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Combined Sewer Overflow Abatement Program Rochester, NY-Volume 1: Abatement Analysis

O'Brien and Gere Engineers, Inc.
Syracuse, NY

Prepared for

Municipal Environmental Research Lab.
Cincinnati, OH

Jul 81

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National Technical Information Service
NTIS

EPA-600/2-81-113
July 1981

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COMBINED SEWER OVERFLOW ABATEMENT PROGRAM, ROCHESTER, N.Y.

Volume I: Abatement Analysis

by

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Grant No. Y005141

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and

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U.S. ENVIRONMENTAL PROTECTION AGENCY
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16. ABSTRACT

Pollution abatement analyses conducted in conjunction with system network modeling studies and supported by combined sewer overflow (CSO) monitoring and sampling were initiated with the ultimate goal of formulating a cohesive and workable Master Plan for CSO reduction and control.

Overflow monitoring and sampling data from thirteen CSO locations within the Rochester, New York Pure Waters District Served as the basis for network modeling studies. The receiving water models were used to evaluate various CSO pollution abatement alternatives. Nonstructural, minimal structural, and structurally intensive alternatives were defined and evaluated by these models. The nonstructural approach centered around the application of Best Management Practices (BMPs), and the structural alternatives involved evaluation of conventional storage and treatment options.

Cost benefit analyses of all structurally intensive alternatives were conducted using optimum treatment process train configurations developed from pilot plant evaluations, as reported in Volume II (EPA-600/2-79-031b, NTIS PB#80-159262) of this project's report.

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a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Combined sewers, *System network models, *Overflows---Sewers, Water pollution, *Mathematical models, Waste storage, Waste treatment, Cost analysis, *Pilot plants, Contaminants, *Cost Effectiveness, Sedimentation, Sewage.	Combined sewer overflows, Pollution abatement, Water pollution control, Rochester, N.Y., Suspended solids removal, *Flocculation/sedimentation, *Swirl degritter, *Swirl primary separator, *Microscreening, *Wastewater treatment, *High-rate dual-media filtration, *Carbon adsorption, *High-rate disinfection, Chlorine dioxide, Alum, Mixing intensity, Storm runoff.	13B

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FOREWORD

The Environmental Protection Agency was created because of increasing public and governmental concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in the problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

The Great Lakes National Program Office, through Section 108(a) of PL 92-500, enters into grants for the demonstration of new methods and techniques and for the development of preliminary plans for the prevention, reduction or elimination of pollution within all or any part of the watersheds of the Great Lakes. The Great Lakes National Program Office has joined with the Municipal Environmental Research Laboratory in carrying out this research and demonstration project to assist the Rochester Pure Waters District to eliminate an urban drainage pollution problem to Lake Ontario.

The deleterious effects of storm sewer discharges and combined sewer overflows on the nation's waterways have become of increasing concern in recent times. Efforts to alleviate the problem depend on characterization of these flows as to both quantity and quality. This report describes the general approaches available for the control of combined sewer overflows and their application to the Rochester, New York combined sewer system.

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Director
Municipal Environmental Research
Laboratory

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Director
Great Lakes National
Program Office

ABSTRACT

Pollution abatement analyses, conducted in conjunction with system network modeling studies and supported by combined sewer overflow (CSO) monitoring and sampling, were initiated with the ultimate goal of formulating a cohesive and workable Master Plan for CSO reduction and control. The Master Plan was developed in light of fiscal constraints, sewer system complexities, necessity for optimized benefits from minimal capital and operating expenditures, and best use policies of the affected receiving waters. The presented methodology is considered applicable to other urban areas.

Overflow monitoring and sampling data from thirteen CSO locations within the Rochester, New York Pure Waters District collected during the period January through December, 1975 served as the basis for network modeling studies. The USEPA Stormwater Management Model - Version II, Simplified Stormwater Model, and receiving water models were used to evaluate various CSO pollution abatement alternatives. Nonstructural, minimal structural, and structurally intensive alternatives were defined and evaluated by these models. The nonstructural approach centered around the application of Best Management Practices (BMPs), and structural alternatives involved evaluation of conventional storage and treatment options.

Cost benefit analyses of all structurally intensive alternatives were conducted using optimum treatment process train configurations developed from pilot plant evaluations, as reported in Volume II of this project's report.

Preliminary analysis of BMP and minimal structural alternatives indicated that by addressing the major sources of pollution and by eliminating throttling constraints within the existing sewerage system, a substantial decrease in the total annual load of contaminants to the receiving waters from rainfall induced CSO can be achieved for relatively small capital expenditures. These measures can be initiated within a short period of time, thereby immediately reducing pollution to the receiving waters while long term design and construction of more structurally intensive alternatives are undertaken.

This Report was submitted in fulfillment of Grant No. Y-005141 by O'Brien & Gere Engineers, Inc. under the partial sponsorship of the U.S. Environmental Protection Agency. Work was completed as of November, 1976.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

ac -- acre
ave. -- average
BMP -- Best Management Practice
BOD or BOD_5 -- 5 day biochemical oxygen demand at 20°C
C -- concentration, units as specified
cfs -- cubic feet per second
CLFREQ -- street cleaning frequency
CSO -- combined sewer overflow
D, dia -- diameter
D_2 -- chamber diameter
DD -- dust and dirt
DIS -- disinfection
DMF -- dual-media filtration
DMHRF -- dual-media high-rate filter
D.T. -- detention time
f. coliform -- fecal coliforms
F-F -- first-flush
F/S -- flocculation/sedimentation
f. strep -- fecal streptococci
ft -- feet
ft^2 -- square feet
G -- velocity gradient (sec^{-1})
g -- grams
gpd -- gallons per day
G/S -- swirl degritter
ha -- hectares
hr -- hour
in -- inch
K -- rate constant or thousand
kg -- kilograms
lb -- lb
m -- meters
m^2 -- square meters
m^3 -- cubic meters
MA7CD10 -- minimum average 7 consecutive day flow occurring every 10 years
mi^2 -- square miles
mil gal -- million gallons
mgd -- million gallons per day
mg/l -- milligrams per liter
min -- minutes

ABBREVIATIONS (Con't)

MPN -- most probable number
NF -- Froude number
no. -- number
O&G -- oil and grease
OR -- overflow rate
P/S -- swirl primary separator
psf -- pounds per square foot
Q_D -- design flowrate
QQS -- quantity-quality simulation
r -- ratio of hour of maximum rainfall to total duration
sec -- second
Set. S -- settleable solids
SOR -- surface overflow rate
SS -- suspended solids
SSM -- Simplified Stormwater Model
std dev -- standard deviation
STP -- sewage treatment plant
SWMM -- USEPA Stormwater Management Model - Version II
t. coliform -- total coliform bacteria
TIP -- total inorganic phosphorous
TKN -- total Kjeldahl nitrogen
trtmt -- treatment
TSS -- total suspended solids
VSS -- volatile suspended solids
v -- velocity
yd^3 -- cubic yards
yr -- year

SYMBOLS

Cl_2 -- chlorine
ClO_2 -- chlorine dioxide
Q -- flowrate, units as specified
$ mil -- million dollars

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This report has been prepared by O'Brien & Gere Engineers, Inc., Syracuse, New York under the direction of Frank J. Drehwing, Vice President, and Cornelius B. Murphy, Managing Engineer.

SECTION 1

INTRODUCTION

BACKGROUND

The significance of pollution resulting from urban storm runoff is well documented. A large portion of this pollution is attributable to combined sewer overflow (CSO) or discharges from relief points within a combined sewer system.

Urban stormwater management is a continuous process. Essential to its success is constant innovation, demonstration, assessment, implementation guidance, and active program feedback (1). Based upon 1980 dollars, the cost of abating pollution from CSO is estimated to be approximately $31 billion (2). Such a program, therefore, must be based on proven capabilities, practical methodology and assessment criteria, an expanding data base, and a continuous technology transfer.

PROBLEM DEFINITION

All programs adopted by Monroe County for the Rochester Pure Waters District are directed at meeting water quality standards as established by state and federal legislation for the Rochester, New York area. These programs specifically address the problem of receiving water quality degradation due to urban storm runoff and subsequent CSO. Figure 1 shows the study area including the major receiving water bodies, intercepting sewer network, and significant overflow relief points.

Present CSO results in the direct contravention of established water quality standards for the Genesee River, imposes heavy nutrient and chemical loadings on Irondequoit Bay, and causes bacterial contamination of the public bathing beaches along the Rochester Embayment of Lake Ontario (3). The latter has resulted in periodic beach closing days during the summer months.

Aside from the direct impacts of CSO, such as objectionable floating material and high bacteria loadings, the overflows contribute to the contravention of stream standards in the lower reaches of the Genesee River under dry-weather conditions. This is caused by the settling of oxygen demanding materials discharged during overflow events.

Previous studies of the District's combined sewer system (4,5,6) cited major deficiencies in the existing sewer system and identified the effects of CSO on the area receiving waters. These studies have all recognized the

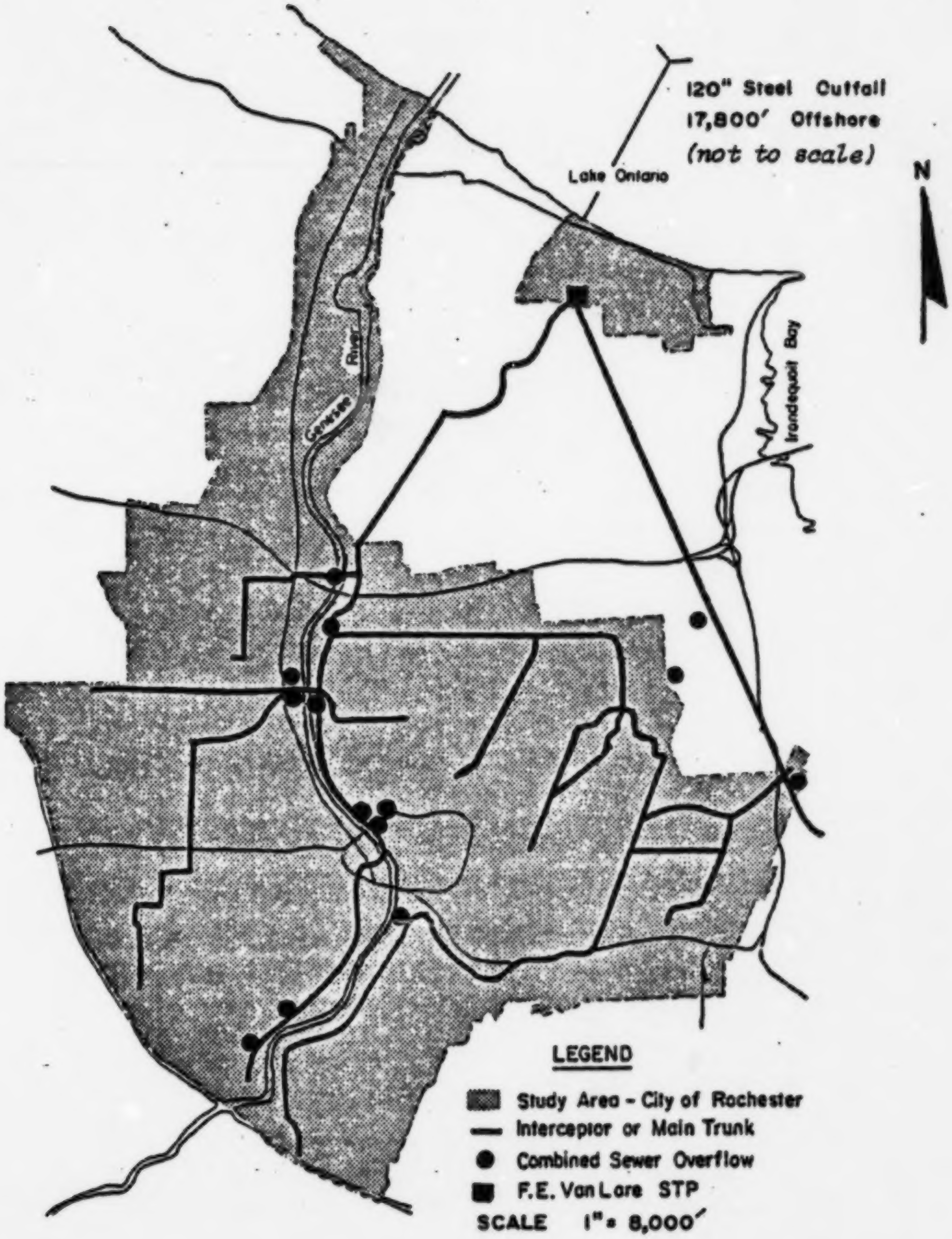
120" Steel Outfall
17,800' Offshore
(not to scale)
Lake Ontario
N
Genesee River
Irondequoit Bay
LEGEND
▨ Study Area - City of Rochester
— Interceptor or Main Trunk
● Combined Sewer Overflow
■ F.E. Van Lare STP
SCALE 1" = 8,000'

FIGURE 1. Study Area

need for measures to collect, control, and treat CSO within the City of Rochester.

PURPOSE OF STUDY

In response to the transient water quality problems induced by periodic overflows from the Rochester Pure Waters District combined sewer system, a Research and Demonstration project was undertaken jointly by the U.S. Environmental Protection Agency (USEPA) and Monroe County, New York. The primary purpose of the project was to develop an abatement and management program necessary to achieve a cost effective solution to the CSO induced impairment of the Genesee River, Irondequoit Bay, and Lake Ontario. In developing such a program, a practical methodology of approach was formulated to analyze, assess, and evaluate CSO pollution and abatement alternatives.

A second purpose was to demonstrate the usefulness of mathematical models simulating both the urban rainfall-runoff process and the subsequent stormwater flows within a large combined sewer system. Specific models that were evaluated include the Simplified Stormwater Model (SSM) developed by Metcalf & Eddy, Inc. (7) and the USEPA Stormwater Management Model-Version II (SWMM).

This Report also presents the merits associated with the implementation of a Best Management Practices (BMP) program to abate CSO. Both source and collection system management options were evaluated. The developed management program including the methodology of approach, the urban stormwater mathematical modeling, and abatement alternative analysis, involving both structural and BMP measures, lead to formulation of a master plan for CSO pollution abatement within the Rochester Pure Waters District. In the process, it was hoped that the resulting developments in the area of treatment technology, CSO monitoring, overflow sampling and characterization, urban runoff modeling, and pollution abatement methodology may help other municipalities define and solve their urban runoff problems.

PROJECT ELEMENTS

To achieve the stated objectives of this study, the Research and Demonstration project was divided into three basic elements: a CSO monitoring and assessment program, a CSO mathematical modeling program, and a pilot plant demonstration program. Figure 2 presents the three basic elements of the project and their relationship to the development of the Master Plan. The monitoring and modeling programs are described in detail in Volume I, whereas, the pilot plant studies are described in detail in Volume II (8). These three elements evolved from the formulation of a methodology of approach to the development of a Master Plan.

As part of the overflow monitoring program, an intensive CSO flow recording and sampling system was implemented to define the frequency, volume, and pollutant characteristics associated with the District's CSO

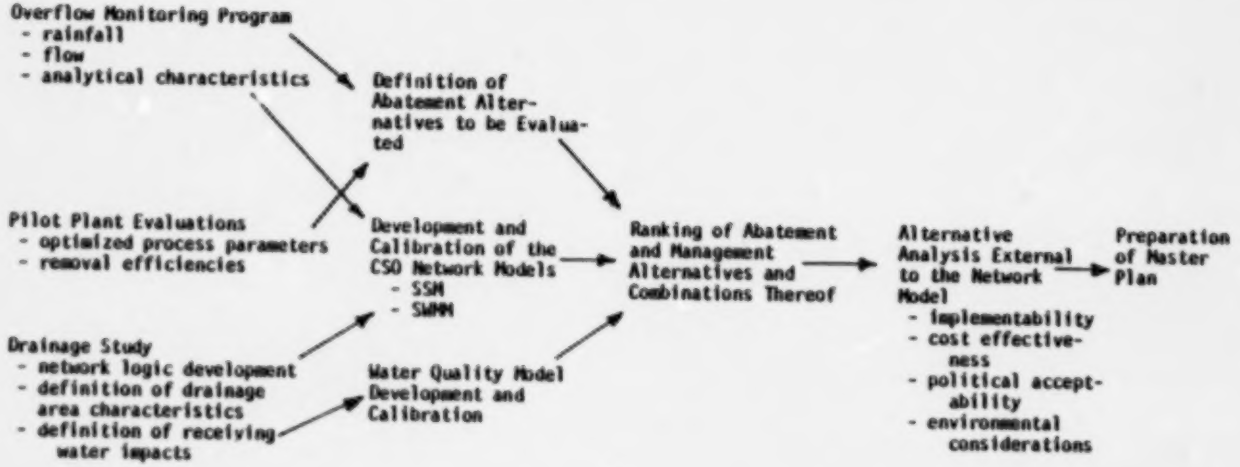
Overflow Monitoring Program
- rainfall
- flow
- analytical characteristics
Pilot Plant Evaluations
- optimized process parameters
- removal efficiencies
Drainage Study
- network logic development
- definition of drainage area characteristics
- definition of receiving water impacts
Definition of Abatement Alternatives to be Evaluated
Development and Calibration of the CSO Network Models
- SSM
- SWMM
Water Quality Model Development and Calibration
Ranking of Abatement and Management Alternatives and Combinations Thereof
Alternative Analysis External to the Network Model
- implementability
- cost effectiveness
- political acceptability
- environmental considerations
Preparation of Master Plan

FIGURE 2. Program Elements and Their Relationship to the Development of the Master Plan

discharges. A drainage basin field investigation was conducted to define those basin parameters that affect the urban stormwater runoff process. These two programs provided the necessary data sets, including representative CSO hydrographs and pollutographs, drainage basin characteristics, and sewer system inventory, to facilitate model calibration and verification. Included in the modeling effort was the refinement and verification of the Genesee River water quality model previously developed by Limno-Tech, Inc. (9). An important task in the modeling program was the preliminary definition of pollution abatement alternatives. Figures 3 and 4 present the tasks associated with the monitoring and modeling programs, respectively.

The pilot plant program involved the design and construction of a pump station and pilot treatment facilities necessary to evaluate the effectiveness of eight unit processes. The evaluated processes included high-rate flocculation/sedimentation, swirl degritting, swirl primary separation, high-rate dual-media filtration, granular activated carbon adsorption, high-rate disinfection using chlorine and chlorine dioxide, and microscreening. The pilot plant results were utilized to develop process models and associated cost effectiveness relationships. Figure 5 presents the detailed tasks associated with the pilot plant efforts.

It must be emphasized that the three major program elements are not independent of each other but are closely interrelated and oriented towards the evaluation of applicable abatement alternatives and the development of the master plan. Figure 6 presents the relationship of each of the outlined program elements to the master plan development for the Rochester Pure Waters District.

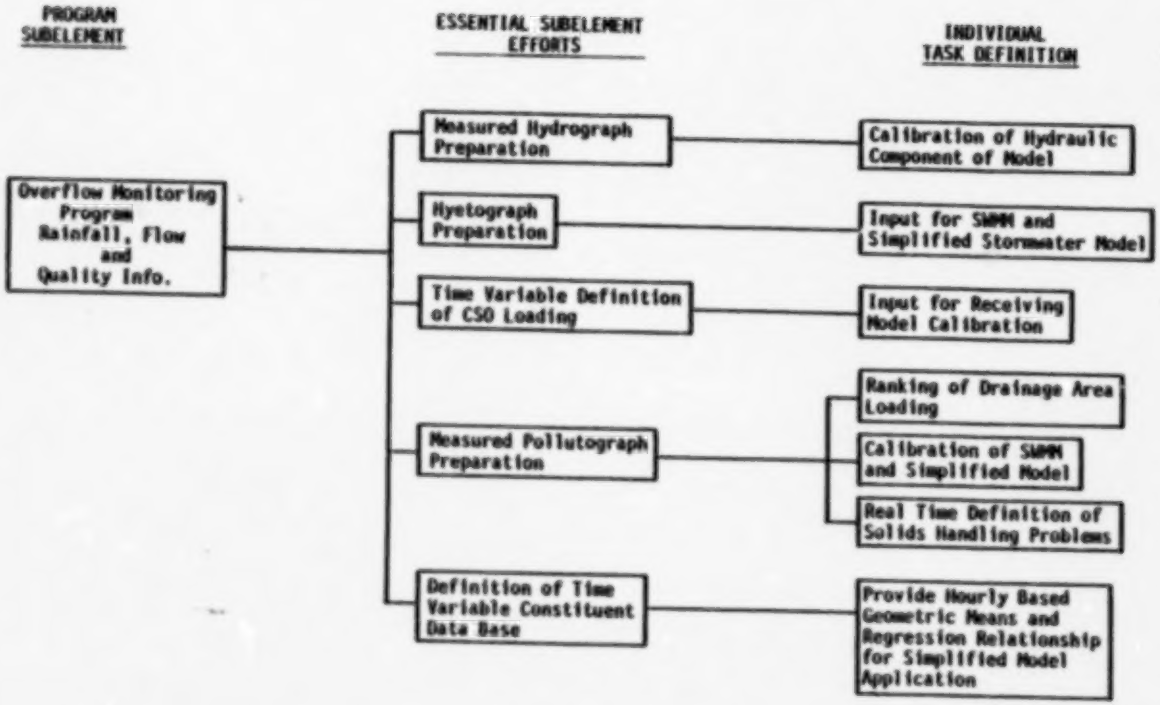
PROGRAM SUBELEMENT
ESSENTIAL SUBELEMENT EFFORTS
INDIVIDUAL TASK DEFINITION
Overflow Monitoring Program Rainfall, Flow and Quality Info.
Measured Hydrograph Preparation
Hyetograph Preparation
Time Variable Definition of CSO Loading
Measured Pollutograph Preparation
Definition of Time Variable Constituent Data Base
Calibration of Hydraulic Component of Model
Input for SWMM and Simplified Stormwater Model
Input for Receiving Model Calibration
Ranking of Drainage Area Loading
Calibration of SWMM and Simplified Model
Real Time Definition of Solids Handling Problems
Provide Hourly Based Geometric Means and Regression Relationship for Simplified Model Application

FIGURE 3. Overflow Monitoring and Characterization Effort

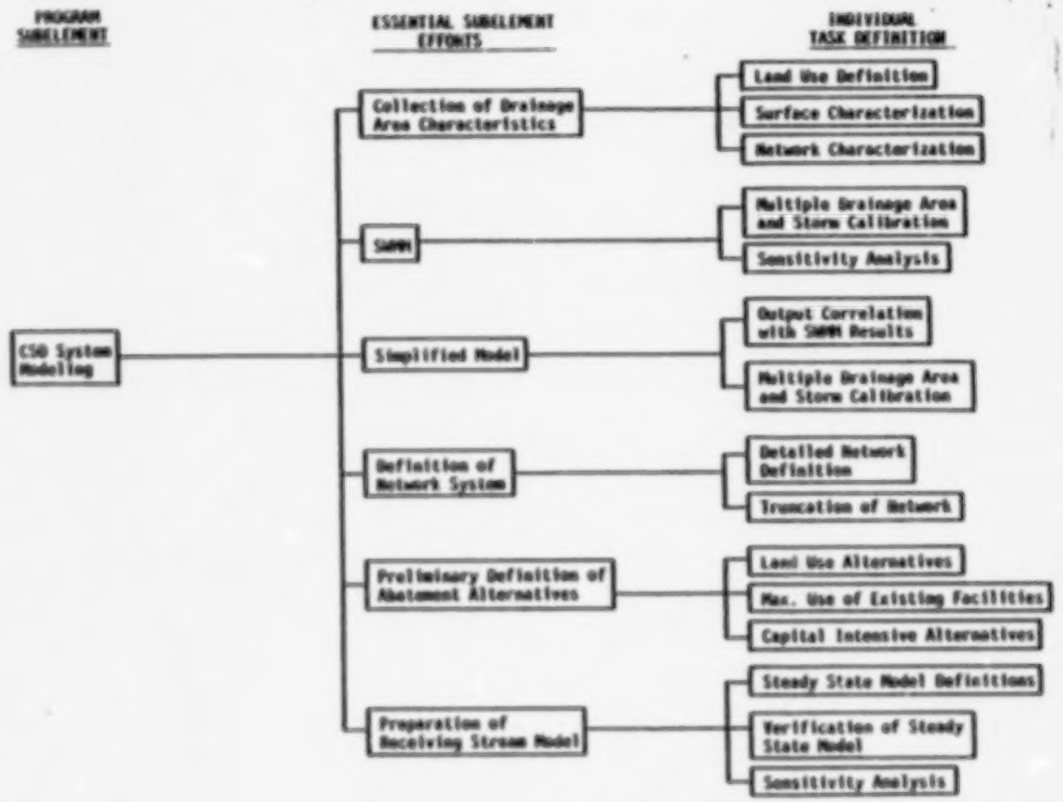
PROGRAM SUBELEMENT
ESSENTIAL SUBELEMENT EFFORTS
INDIVIDUAL TASK DEFINITION
CSO System Modeling
Collection of Drainage Area Characteristics
Land Use Definition
Surface Characterization
Network Characterization
SWMM
Multiple Drainage Area and Storm Calibration
Sensitivity Analysis
Simplified Model
Output Correlation with SWMM Results
Multiple Drainage Area and Storm Calibration
Definition of Network System
Detailed Network Definition
Truncation of Network
Preliminary Definition of Abatement Alternatives
Land Use Alternatives
Max. Use of Existing Facilities
Capital Intensive Alternatives
Preparation of Receiving Stream Model
Steady State Model Definitions
Verification of Steady State Model
Sensitivity Analysis

FIGURE 4. Combined Sewer Overflow System Modeling Effort

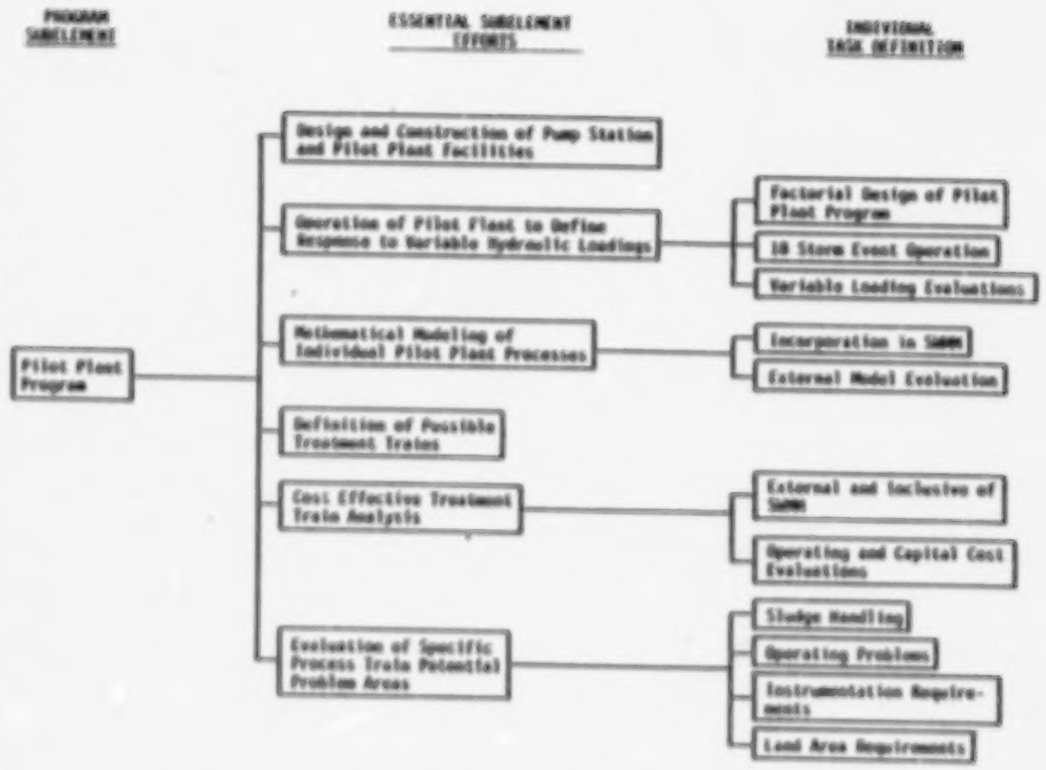
PROGRAM SUBELEMENT
ESSENTIAL SUBELEMENT EFFORTS
INDIVIDUAL TASK DEFINITION
Pilot Plant Program
Design and Construction of Pump Station and Pilot Plant Facilities
Operation of Pilot Plant to Define Response to Variable Hydraulic Loadings
Mathematical Modeling of Individual Pilot Plant Processes
Definition of Possible Treatment Trains
Cost Effective Treatment Train Analysis
Evaluation of Specific Process Train Potential Problem Areas
Factorial Design of Pilot Plant Program
10 Storm Event Operation
Variable Loading Evaluations
Incorporation in SWMM
External Model Evaluation
External and Inclusive of SWMM
Operating and Capital Cost Evaluations
Sludge Handling
Operating Problems
Instrumentation Requirements
Land Area Requirements

FIGURE 5. Pilot Plant Effort

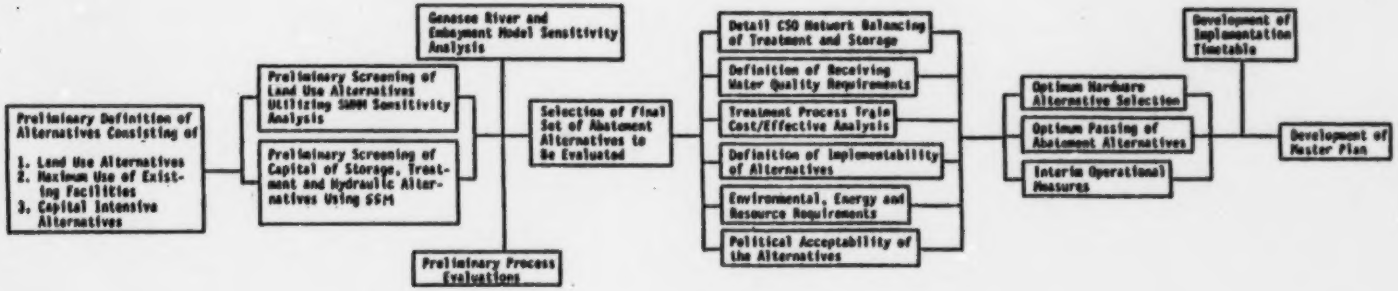
Preliminary Definition of Alternatives Consisting of
1. Land Use Alternatives
2. Maximum Use of Existing Facilities
3. Capital Intensive Alternatives
Preliminary Screening of Land Use Alternatives Utilizing SWMM Sensitivity Analysis
Preliminary Screening of Capital of Storage, Treatment and Hydraulic Alternatives Using SSM
Genesee River and Embayment Model Sensitivity Analysis
Preliminary Process Evaluations
Selection of Final Set of Abatement Alternatives to Be Evaluated
Detail CSO Network Balancing of Treatment and Storage
Definition of Receiving Water Quality Requirements
Treatment Process Train Cost/Effective Analysis
Definition of Implementability of Alternatives
Environmental, Energy and Resource Requirements
Political Acceptability of the Alternatives
Optimum Hardware Alternative Selection
Optimum Passing of Abatement Alternatives
Interim Operational Measures
Development of Implementation Timetable
Development of Master Plan

FIGURE 6. Evaluation Process for Abatement Alternative Analysis

SECTION 2

CONCLUSIONS

Based on the results of this investigation, the following conclusions relative to CSO management and abatement are presented:

General

1. A rigorous definition of the existing system of CSO and stormwater facilities is fundamental to the development of an abatement program. This definition includes the identification of major drainage basins, major trunk and intercepting sewers, and CSO and stormwater relief points.

2. The installation and proper maintenance of overflow monitoring instrumentation are essential for both receiving water problem definition and any subsequent sewer network and water quality model calibration and verification.

3. Accurate rainfall data collection and subsequent statistical analyses, including design storm definition and formulation, are essential in evaluating the response of the existing system as well as the effectiveness of various abatement alternatives.

4. Development of a methodology of approach and definition of applicable abatement alternatives early in the program insure that the purpose of the study is not lost and all data collection activities are conducted according to the required analyses.

5. SSM is capable of providing a preliminary screening of potential abatement alternatives involving a balance between storage and treatment.

6. SWMM (USEPA Stormwater Management Model-Version II) can project the urban storm runoff and quantities within acceptable confidence limits but is presently limited in its ability to simulate overflow quality.

7. Overflow quality can be better simulated through the application of statistical techniques using actual monitored overflow data.

8. The ability to abate CSO pollution resulting from an infrequent design storm event will require the implementation of structurally intensive facilities, minimal structural improvements to the existing sewer system, or the implementation of nonstructural abatement alternatives, known as BMPs.

Specific to Rochester, New York

1. Hydraulic simulations on the existing sewer system, based on the application of a simplified stormwater model, indicate that overflows occur, on the average, 66 days annually. The total volume associated with these overflows discharged to all receiving waters within the Rochester Pure Waters District amounts to 1.9 billion gallons.

2. Because most CSO are generated within highly developed urban areas, implementation of large volumes of surface storage that could alleviate some of the problems is economically impractical.

3. Point source treatment for CSO is infeasible due to size requirements and field limitations. The only point source treatment alternative investigated was the application of swirl primary separators installed at the discharge point of the combined sewers. Results obtained from the Pilot Plant Studies indicate that for the high rates of flows associated with these overflows, relatively large diameter separators (approximately 100 ft) would be required. Available land adjacent to the overflow locations for implementation of these large separators is nonexistent.

4. Additional studies relative to both soil conditions and solids removal are required to properly assess the feasibility of underground storage facilities.

5. More intensive street cleaning can alleviate a portion of CSO induced water quality degradation. Although many of the surface quality routines associated with SWMM could not be verified, sensitivity analyses conducted on the model indicate that increased street sweeping can reduce the total load of contaminants reaching the sewer system.

6. More intensive sewer maintenance can help alleviate CSO induced deterioration of receiving water quality. Review of sewer maintenance programs recently adopted by Monroe County indicates that the number of flooding complaints received during a storm event can be reduced by additional sewer cleaning.

7. Additional solids investigations are necessary to determine the feasibility of using porous pavement instead of conventional asphalt to decrease the rate of urban stormwater runoff. Preliminary modeling results indicate that the rate of runoff can be substantially reduced upon application of porous pavement throughout the City. Institutional constraints, however, may limit the implementability of porous pavement on a large area-wide basis.

8. Disconnecting roof drains from the sewer system can significantly mitigate the volume and frequency of flooding in selected urban areas. The effect on CSO has not yet been determined. Institutional constraints and public nonparticipation may be strong deterrents to implementing roof downspout disconnection ordinances.

9. The minimal structural abatement alternatives that can significantly reduce CSO pollution include: selective upgrading of the present interceptor system by removing three throttling constraints, selective overflow weir/regulator modifications, and blockage of high-impacting overflows.

   The three throttling constraints are along the St. Paul Boulevard Interceptor and specifically include:

   (1) the siphon across the Genesee River conveying flows from the Lexington Avenue and West Side Trunk sewers to the interceptor,

(2) the section of interceptor from the Carthage Avenue junction chamber to the junction chamber at Norton Street and St. Paul Boulevard, and

(3) the siphon that is part of the interceptor conveying flows from the west side to the east side of the Genesee River.

Selective overflow weir/regulator modifications include raising of present weir level settings and altering present gate and flow settings within several overflow regulators.

Blockage of several high-impacting overflows can reduce the total pollutant load presently being discharged to the Genesee River. Additional detailed hydraulic modeling is necessary to determine the possibility of any adverse surcharge conditions existing within these sewers after blockage that could cause flooding problems.

10. On the basis of projections using a simplified mathematical stormwater model, the non- and minimal structural abatement alternatives are expected to reduce significantly the existing volume of CSO, and the average annual BOD_5 and TSS loadings to the Genesee River.

11. The capital costs associated with the recommended CSO pollution abatement measures involving non and minimal structural alternatives are estimated at 12 million dollars (1976 dollars).

12. In Rochester, one structural alternative involves grit removal, in conjunction with the optimized operation of the F. E. Van Lare Treatment Facility.

13. The very low assimilation capacity of the Genesee River, as indicated from previous water quality investigations, especially during the months of June, July, August, and September, precludes the construction of a centralized overflow treatment facility along the Genesee River.

14. Additional control structures within the present sewer system would be necessary to utilize the large volume of potentially available in-system storage capacity.

15. In the interim, during which the structurally intensive abatement alternatives are designed and constructed, application of various BMPs can significantly reduce the annual CSO pollutant loading to the Genesee River. These measures include selective overflow weir and regulator modifications or adjustments, removal of several conveyance constraints within the St. Paul Boulevard Interceptor, and implementation of inflow restriction regulations such as use of porous pavement in select areas.

SECTION 3

RECOMMENDATIONS

It is recommended that:

1. Familiarization with the various mathematical models for urban storm runoff simulations be made before application of a specific model. This will insure that the model best suited for a defined objective will be selected.

2. Network models, such as SWMM (USEPA Stormwater Management Model-Version II), be relied upon for determining runoff and overflow volumes for selected storm events and less for estimating the quality of the runoff. Estimates of surface runoff quality should be made using statistical analyses of actual field monitored data.

3. Initial screening, planning, and design of storage and treatment abatement alternatives be made with a simplified continuous simulation model. Computer costs associated with many detailed hydraulic models can be prohibitive in many cases; therefore, only a model that will satisfy the objectives of a study at the least possible cost should be utilized.

4. Hydraulic analysis and design of sewer systems be conducted with a detailed network model such as SWMM.

5. Rainfall characterization be based primarily on the use of historical precipitation data, although the design storm approach may have to be applied in certain situations. More research must be conducted on the concept of design storms to establish design rainfall hyetographs that could be applied with mathematical network runoff models.

6. Models not be used to predict runoff/overflow quantities or qualities without proper field calibration and verification. A relatively detailed field monitoring program is essential in providing the background data for proper model calibration and verification.

7. Continued research and development be made in the area of flow measuring devices for sewer applications, especially under wet-weather flow conditions. Field conditions should dictate the type of monitoring device to be installed.

8. More statistical analyses be conducted to better establish the correlation between runoff quality and parameters such as rainfall and land use characteristics.

9. Field monitoring systems, regardless of the type and sophistication, be frequently inspected and verified on a weekly basis to insure proper data collection.

10. Additional detailed hydraulic analyses be conducted to better define the interceptor throttling constraints, regulator/weir modifications, and control structure locations. The SWMM is capable of providing the required analyses.

11. Additional studies be conducted on the identification of dissolved oxygen impacts on the Genesee River resulting from CSO discharges. Previous water quality investigations have indicated that the lower reaches of the river became most severely impacted by CSO discharges relative to dissolved oxygen concentrations. An automated water quality monitor should be located within the lower reach of the Genesee River and operated over a period of one year to best define water quality degradation due to these CSO discharges.

12. BMPs be considered when conducting any CSO abatement program. In many instances implementation of BMPs, possibly in conjunction with minimal-structural alternatives, can alleviate many problems associated with frequent CSO discharges. Failure to investigate their effect could severely limit establishing cost effective abatement solutions.

13. A program to investigate the effectiveness of increased street sweeping on reducing the pollutant loadings to the sewer system be initiated. Such a program should include provisions for correlating the effects of increased street sweeping operations, types of equipment used, street parking use and restrictions, and program costs, with reduction of surface pollutants available for washoff during storm events.

14. Closely associated with the street sweeping program should be a study to investigate the cost effectiveness of increased sewer flushing and sewer system maintenance in abating CSO pollution.

15. Control structures be installed at selected locations within the existing and proposed sewer system. The exact location of these structures should be determined by conducting additional detailed hydraulic modeling with the use of a simulation model such as the SWMM. The type of structure to be implemented should be determined by actual field conditions in conjunction with results of the network modeling.

16. The development and implementation of a Master Plan for CSO pollution abatement follow a sequence of phasing of required system improvements according to their projected cost-effectiveness.

17. Scheduled reviews be included in any CSO abatement program to periodically evaluate the effectiveness and cost/benefits associated with alternative implementation. This periodic review will insure that previously defined objectives are being met and, if not, changes to the program can be made to better solve the initial problems.

SECTION 4

METHODOLOGY OF APPROACH TO ALTERNATIVE ANALYSIS

GENERAL METHODOLOGY

In the process of conducting the Rochester CSO study a methodology for the analysis of CSO abatement alternatives was developed. The abatement alternative analysis involved the logical development of a comprehensive data base and the subsequent analysis activities involving fifteen major activity categories herein described.

A number of points were inherently stressed in the development of the methodology sequence. The first of these guiding points was the need to define the applicable abatement alternatives very early in the program. This process helped insure that the purpose of the study was frequently refocused and all data gathering activities would be conducted according to the required analyses.

The importance of installing an accurate, reliable, and easily maintained field monitoring and sampling system cannot be overemphasized. The collection of accurate characteristic overflow data is paramount in defining the CSO pollutant loadings to the receiving waters and in establishing the required model calibration and verification data sets.

Another important step is model calibration and verification. The models selected for application were based on the urban area and collection system requirements as well as on the objectives defined in the initial phase of the program. The Simplified Stormwater Model was used in the initial screening of alternatives, allowing extensive system analyses to be conducted utilizing minimal computer time. The final analysis was conducted on a select number of abatement alternatives with the use of SWMM*. In conducting the network modeling activity it became apparent that one should not use models which are more complex than the analysis requires. One should utilize the simplest model which will attain the defined objectives. It is for this reason that the Simplified Stormwater Model was utilized in the initial screening of alternatives while the use of SWMM was utilized in the final analysis of a select number of abatement alternatives.

The following major tasks were considered essential for the development of the Master Plan:

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*The exact modeling framework is discussed in detail in Section 6.

1. Definition of Program Objectives
    a. minimize the wet-weather discharge of combined sewer overflows to the receiving waters
    b. maximize the effective utilization of existing facilities
    c. develop cost-effective alternative solutions
    d. develop solutions which minimize the energy and resource consumption

It is essential in conducting a comprehensive combined sewer overflow abatement alternative analysis program that the program objectives be well defined early in the analysis. It is also important that the program objectives be frequently reviewed in the course of conducting the analysis so that the tools and their attendant complexity do not become the objective rather than the means of attaining the objective.

It is also of tremendous importance that the optimum utilization of existing conveyance and treatment facilities be identified early on as a fundamental objective. There are many collection and treatment system management options that can be employed to minimize the discharge of combined sewer overflows. In seeking cost-effective solutions to wet-weather discharges, the full and optimum utilization of the existing facilities should be a high priority program objective.

2. Definition of Existing Conveyance and Treatment Systems
    a. dry-weather conveyance and treatment capacity
    b. dry-weather treatment efficiency
    c. wet-weather conveyance and treatment capacity
    d. wet-weather treatment efficiency
    e. system wet-weather relief points
    f. system control measures
    g. separate sewered areas

The development of an extensive data base and understanding of the operation and response of the existing conveyance and treatment system is required before any system modeling can be conducted. For that matter a basic understanding of the function and present utilization of the existing facilities is required before the appropriate collection system model can be selected.

It is particularly important that the available drawings for the conveyance system, regulating structures, and control structures be reviewed and compared to the conveyance system as built. A field survey of all wet-weather relief points on the system is most important. The Monroe County Division of Pure Waters had maintained good documentation; however, this is found not to be the case with many metropolitan areas.

A beneficial activity in defining the available treatment capacity involves the analysis of the treatment efficiency as a function of plant inflow and total precipitation. Conducting this type of analysis fully introduces the investigators to the limitations of the available data base as well as the facilities.

3. Definition of Drainage Areas
   a. division of service district into relief point catchment areas
   b. determination of area and land use characteristics of the defined catchment areas
   c. determination of percent imperviousness
   d. establishment of dry-weather quantity/quality characteristics

The delineation of the combined sewered service area into discrete hydrologic catchment areas is required for even the most basic form of system analysis. This can be accomplished by reviewing the available collection system and topographic mapping for the study area. Each of the delineated catchment areas must be associated with the system overflow points to which it is tributary.

Once the catchment areas have been delineated, basic surface definition is required. The land use, population density, and percent imperviousness must be established for each catchment area. Associated collection system dry-weather flow and wastewater characteristics are also required as input to the network modeling activity.

4. Review of the Meteorological Data Base
   a. extent of historical precipitation record
   b. interval of precipitation records
   c. determination of basic precipitation frequency and magnitude statistics

The most important factor in the generation of pollutant loadings from combined sewer overflow discharges is rainfall; and as such, rainfall data in the form of historical and current precipitation records should be one of the first items obtained and analyzed. Depending on the level of refinement which might be employed in the definition of transient stormwater loads, substantially different rainfall data are required. For preliminary abatement analyses utilizing a simplified stormwater model, precipitation records that may be obtained from the National Weather Records Center in Asheville, North Carolina are usually sufficient. These data are hourly rainfall and snowfall records and collected over long periods of time, usually 20 years or more. Utilization of more sophisticated hydraulic models will require rainfall data on a much shorter time interval, usually on the order of minutes.

Since the basic driving force in the generation of stormwater pollutant loads is rainfall, determination of precipitation frequency and magnitude statistics is essential for analyzing such stormwater loads, their impacts, and various alternative control strategies. A statistical assessment of such rainfall parameters as intensity, duration, volume, and time between successive events should be developed. This determination can most easily be accomplished through the application of hourly rainfall records supplied by the U.S. Weather Bureau.

5. Selection of Detailed Network Model to be Utilized in the Hydraulic Analysis of the Existing System and the Proposed Alternatives
   a. evaluate control strategies on the basis of single event and multiple event network models
   b. determine need to evaluate surcharge conditions
   c. determine the input data set requirements of the selected model

The various identified control options must be evaluated on the basis of both single event and multiple event network models. Preliminary design of certain structurally intensive alternatives such as overflow treatment plants and storage facilities requires the utilization of single event models to properly size the particular structure. In general, these facilities will have to be designed on the basis of one particular storm event such as a 5 year rainfall. In order to properly evaluate the effectiveness of any implemented structurally intensive alternative, it is essential that such a structure be stressed under both varying rainfall conditions and under the application of successive events. Multiple event models are best suited to accomplish this task.

There are many stormwater models available that evaluate surcharge conditions within the collection system. Depending on the sewer system in question, surcharge conditions may or may not exist within the various sewer conduits within a drainage area. Whether they exist can best be determined from sewer surveys and an overflow monitoring program. The need to utilize a model with surcharge analysis capability is only necessary if field investigations support the presence of surcharge conditions. One should be careful not to utilize a more sophisticated hydraulic model than is actually needed to satisfactorily evaluate the existing conditions.

Many of the available detailed hydraulic models require very large and intensive input data sets for proper utilization. In applying any of these models it is important to initially determine all such requirements before selecting a particular model.

6. Initiation of Overflow and Meteorological Monitoring Program Necessary to Augment the Existing Data Base
   a. hydrograph measurement
   b. pollutograph measurement
   c. characterization of precipitation event intensity/duration
   d. measurement of overflow data on 25 percent of the overflows for six storm events
   e. establishment of statistical relationships for quality parameters

In general, municipalities experiencing a combined sewer overflow problem lack the necessary overflow and meteorological monitoring data. Thus, in order to assess the impact of urban runoff on the receiving waters and to evaluate various control strategies, it is necessary to augment the existing data base. The most important parameters that should be locally monitored include: in-system flows at various locations, major overflow locations, and rainfall data. In addition to these measurements it is essential that the concentrations associated with the overflow. In this manner the total mass loading of contaminants discharged to the receiving waters can be computed.

Flow monitors installed on the overflow conduits indicate the storm induced overflow hydrograph. Knowing the pollutant concentrations developed from the sampling program, pollutographs can be established. Local raingauges located throughout the study area characterize the rainfall in terms of duration, intensity, and volume. This information provides some of the necessary input data required for all hydraulic network models. Statistical relationships can then be developed that relate rainfall and the overflow volumes and quality. These relationships are essential in evaluating various control strategies and abatement alternatives.

Since monitoring of the overflows and rainfall can continue indefinitely upon initial installation of the monitoring equipment, the question becomes how many monitored storm events constitute a statistically reliable data base on which to project future loadings resulting from implementation of various abatement measures. It is believed that reasonable overflow data collected for six storm events from 25 percent of all major CSO discharge locations can provide sufficient data for subsequent abatement alternative analyses. The resulting data base should allow for an adequate span in monitored land use characteristics and population densities for the total study area. Variations from this rule of thumb are frequently encountered in smaller systems and in systems having complex interconnections.

7. Establish Relevant Abatement Alternatives
   a. Best Management Practices
      Source Management
         street cleaning
         sewer flushing
         catchbasin cleaning
         improved waste collection
         stock pile covering
         limitation of percent imperviousness through zoning
         porous pavement application

      Collection System Management
         detention basins
         playground or parking lot temporary storage
         roof top storage
         optimization of in-system storage
         addition of polymers
         improved system regulation
         inflow/infiltration reduction

   b. Structural Intensive Alternatives
      Storage
         in system storage
         off-line storage (caverns, lagoons, structures)
      Treatment
         high-rate physical/chemical processes
         multiple use dry-weather/wet-weather facilities

c. Optimization of the Existing Facilities
    - removal of conveyance system throttling constraints
    - optimization of treatment processes

Although no single abatement alternative is uniquely appropriate for all municipalities with CSO problems, it is necessary to establish and identify those relevant alternatives early in any urban stormwater assessment. The methodology to be applied in such a study is presented herein and is dictated by many factors. These factors include:

a) local conditions
    - land use
    - conveyance system
    - rainfall patterns

b) availability of local data
    - receiving water quality
    - collection system definition
    - rainfall
    - demographic

c) type of receiving water body

d) specific water quality problems
    - dissolved oxygen
    - bacteria
    - solids
    - floatables

e) time scale of receiving water problems

f) decisions involved
    - feasibility
    - economic risk
    - environmental risk

g) study constraints
    - time
    - costs

The early identification and preliminary assessment of proposed abatement alternatives can limit the amount of time and money subsequently devoted to specific alternative evaluations that are not feasible for one or several reasons. It is advisable that in any abatement program the applicability of implementing Best Management Practices (BMPs) be investigated. BMPs can almost be universally applied at minimal cost within a relatively short period of time. In many instances, evaluation of collection system management BMPs can be accomplished with the use of simplified stormwater models as opposed to the use of more detailed and costly hydraulic models which are necessary to properly evaluate storage and treatment options.

Optimization of existing facilities should also be one of the first alternatives considered under any CSO abatement program. Many CSO induced receiving water impacts result from relatively minor problems within the overall conveyance and treatment system of large urban centers. Generally monies expended to correct these minor deficiencies are well spent in that significant CSO loading reductions can be attained at minimal expense. Detailed evaluation of structurally intensive alternatives involving storage and treatment on a massive scale need be conducted only upon the conclusion that implementation of BMPs and optimization of existing facilities cannot fully realize stated receiving water quality objectives.

8. Use of a Preliminary Screening Model to Evaluate the Existing System - Long Term Simulation
    a. calibration of model on the existing data base
    b. prediction of the annual runoff volume
    c. determination of the annual overflow volume
    d. determination of the frequency of overflow events

The initial step in evaluating any proposed abatement alternative is the determination of existing conditions on an average annual basis. Use of a simplified stormwater model is under long term simulation ideally suited for such an analysis. If no reasonably accurate data base exists, then real monitored data must be obtained for the required calibration of the model. Once the annual volume and frequency of CSO discharges are determined, then the magnitude of the problem and necessary methodology can be established.

9. Initial Evaluation of Storage/Treatment Capital Intensive Alternatives Using a Preliminary Screening Model
    a. establish the preliminary capital intensive alternative configurations for the range of alternatives being evaluated
    b. determination of expected reduction of annual overflow volume

Since most of the detailed hydraulic models are prohibitively costly for long term abatement evaluations, initial evaluation of structurally intensive storage/treatment options should be conducted using a simplified stormwater model. If these evaluations indicate the feasibility of the storage/treatment option in terms of overflow reduction, then more detailed analyses can be conducted to better define the exact option to be implemented.

10. Application of the Detailed Network Model to the Preliminary Evaluation of Structural Alternatives
    a. determine sensitivity response of CSO loading to the application of selected structural alternatives
    b. determine sensitivity response of drainage area characteristics to the application of selected structural alternatives

Due to the cost involved in computer analyses of conveyance systems using a detailed hydraulic model, it is important to initially determine the applicability of these models to the proposed structural alternatives from the standpoint of sensitivity responses to various critical design parameters. An initial evaluation of the modeled response to increases in conveyance capacity, in-line storage capacity, off-line storage capacity or treatment rates can be used to conduct the preliminary abatement alternative analysis. A generalized sensitivity analysis of this nature can allow a number of alternatives to be deleted from additional costly evaluation.

11. Calibration and Verification of the Detailed Network Model
    a. 25 percent of all drainage areas
    b. three to six storm events
    c. verify interceptor flow
    d. compare model projections to those obtained using the Simplified Model output

It is absolutely essential that any applied detailed network model be calibrated and verified before accepting any overflow projections as reasonable. Without having a verified model, there is no way in which to insure the accuracy of the results. Since for any large urban area, calibration and verification of detailed network model runs for all defined drainage areas would be very costly in terms of computer time and preparatory work to define input parameters, it is advised that approximately only 25 percent of all such areas for three to six storm events be evaluated. The selection of those drainage areas should be made on the basis of incorporating a representative sample of all varying land uses, population, percent imperviousness, and acreage associated with these drainage areas. It is important that model projected overflow volumes and frequencies be compared to any existing data base and also to projections from a simplified stormwater model.

12. Selection and Verification of Wet-Weather Quality Predictive Models
    a. selection of water quality models
    b. measurement of wet-weather loading and receiving water quality response
    c. calibration and verification of water quality models
    d. determination of sensitivity of water quality response to wet-weather loading

Since the basic problem in assessing urban stormwater runoff is not specifically the volume, frequency and quality of the runoff but rather the impact it has on receiving water quality, selection and verification of appropriate water quality models are absolutely essential. The subsequent abatement alternative evaluations and final alternative selection will depend almost entirely on the accuracy and reasonableness of the water quality model utilized.

13. Pilot Plant Evaluations of Applicable Treatment Processes
    a. establish the significant process variables
    b. establish process efficiency mathematical models
    c. determine the cost-effective relationship for the preferred process treatment trains

In order to properly assess varying treatment options it is necessary to develop mathematical models that accurately reflect various treatment processes. If existing data are not available for applicable selected treatment options to be investigated, then pilot plant studies should be developed to evaluate all such processes. For the Rochester CSO study such pilot plant studies and developed performance relationships are described in detail in Volume II (8).

14. Detailed Analysis of the Prime Alternatives
    a. determine the optimum balance between storage and treatment under the application of a range of design storms
    b. establish the effectiveness of a potential BMP Program
    c. establish the preferred system relief volumes and their impact on the receiving water body as a function of the design storm frequency
    d. establish the marginal cost to marginal benefit relationship for the prime alternatives
    e. determine the institutional and environmental constraints which may inhibit the implementation of any of the prime alternatives

Since it is extremely difficult, if not impossible, to establish a design storm hyetograph that is utilized by all detailed hydraulic models, it is important that a range of storms be evaluated during abatement alternative analysis. The selection of these storms can be made on the basis of selected historical storm events or on application of several hydrologic methods presently available. The important aspect is that in the process of developing design storm events, consistent reasoning is used.

Important points that are sometimes overlooked are the institutional and environmental constraints associated with any of the prime abatement alternatives. An excellent alternative that is either politically infeasible or which is environmentally unsound cannot be seriously considered for implementation. The institutional and environmental constraints must be considered early in the planning process if a successful program is to be developed. This can be accomplished by bringing the environmental assessment and public participation activities into the mainstream of the planning process.

15. Development of the CSO Abatement Master Plan
    a. develop implementation schedule for the cost-effective solutions
    b. emphasize initial optimization of the existing system
    c. develop cash flow requirements
    d. project receiving water benefits associated with implementation

Any combined sewer overflow study must involve the full development of each of the preceding 14 steps in order to arrive at a cost-effective abatement program which is represented by the formulation of a Master Plan. The developed Master Plan should present the implementation schedule to be followed, emphasize the advantages of early implementation of BMPs and minimal improvements to the existing system's conveyance and treatment. The successful Master Plan should also present cash flow requirements for funding needs, and project the favorable receiving water quality improvements as a result of alternative implementation.

The fifteen tasks presented as essential for the development of a Master Plan for combined sewer overflow abatement have been completed under the various activities which constitute the combined sewer overflow abatement planning for Rochester. The set of guidelines represent a composite of activities developed from both hindsight and foresight, successful initial planning and necessary backtracking. The listing is presented as a working practical guide which would have to be adapted for the individual needs of each planning area.

TECHNICAL PROCEDURES

There is a wide range of technical procedures which have been defined as necessary to develop a combined sewer overflow Master Plan. Many of these procedures have been outlined in the previous section. It is the expressed purpose of this section to outline the use and interrelationships of a number of the mathematical models utilized in the development of the combined sewer overflow abatement program for the City of Rochester. These are also considered applicable to other large urban centers.

Figure 7 shows the relationship between the various models utilized in conducting the required analysis. It is recommended that a simplified stormwater model such as the Simplified Stormwater Model (SSM) be utilized as the screening network model prior to the utilization of a detailed hydraulic model such as the SWMM. The required input data sets to the SSM involve land use information, historical rainfall data, CSO wastewater characterization information, and a defined network configuration.

Upon initial screening of storage and treatment alternatives utilizing the SSM, the use of the more complex SWMM is recommended to obtain a more refined fix of the optimum alternatives. In the Rochester application, the Water Resources Engineers, Inc. (WRE) transport routine was utilized in place of the normal SWMM transport routine. The WRE transport routine has the capability to define areas of potential flooding by means of the backwater and surcharge analysis. These new routines are included in the November 1977 release of the SWMM.

Input data sets required for the application of the SWMM in addition to that utilized by the SSM include dry-weather and industrial flows as well as infiltration/inflow (I/I) data. A more detailed definition of land use as well as a more finely defined sewerage network is also required for the optimum use of the SWMM.

Several process cost/benefit models are recommended to be utilized in the evaluation of the treatment alternatives. These models are extensively described in the pilot plant evaluations for the Rochester, New York CSO Abatement Program (8). Required input information includes the treatment train definition, development of area specific weighting factors, and critical CSO wastewater characterization information. The storage and treatment cost benefit models are then meshed with a cost optimization model to yield the optimized alternative configuration.

The network model output in conjunction with the process cost/benefit model must be analyzed in concert with the output from the water quality models to establish the program cost-effectiveness. The output of the

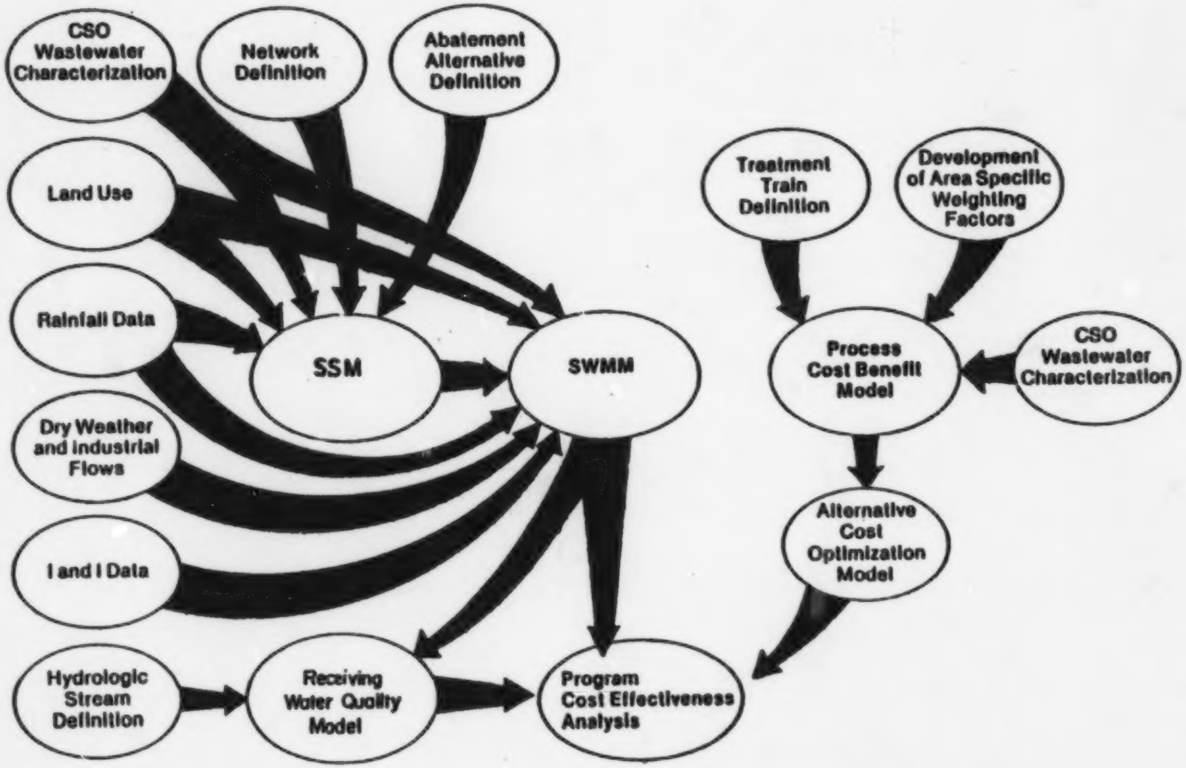

CSO Wastewater Characterization
Network Definition
Abatement Alternative Definition
Land Use
Rainfall Data
Dry Weather and Industrial Flows
I and I Data
Hydrologic Stream Definition
SSM
SWMM
Receiving Water Quality Model
Program Cost Effectiveness Analysis
Treatment Train Definition
Development of Area Specific Weighting Factors
Process Cost Benefit Model
CSO Wastewater Characterization
Alternative Cost Optimization Model

FIGURE 7. Diagram for Model Utilization

modeling effort then assists in the selection of an optimum solution or series of solutions necessary to eliminate the discharge of untreated wastewater from the combined sewer system under investigation. It must be understood that the modeling activity is not an end in itself, but rather an aid in the complex process of developing a Master Plan.

Following is a brief description of each of the modeling tools and their operational requirements:

EPA Stormwater Management Model (SWMM)

A comprehensive mathematical model, capable of representing urban stormwater runoff, has been developed to assist administrators and engineers in the planning, design, evaluation, and management of overflow abatement alternatives (10).

Hydrographs and pollutographs (time varying quality concentrations or mass values) are generated for real storm events and systems from points of origin to points of disposal (including travel in receiving waters) with user options for intermediate storage and/or treatment facilities. Both combined and separate sewer systems may be evaluated. Internal cost routines and receiving water quality output assist in direct cost/benefit analysis of alternate programs of water quality enhancement (11). Efforts on modification and improvement of the SWMM have continued since its initial release.

Simplified Stormwater Model (SSM)

A simplified stormwater model was developed under this program to provide an inexpensive, flexible tool for planning and preliminary sizing of stormwater facilities (7). The model delineates a methodology to be used in the management of stormwater. It consists of a series of interrelated tasks that combine small computer programs and hand computations. The model successfully introduces time and probability into stormwater analysis, promotes total system consciousness on the part of the user, and assists in establishing size effectiveness relationships for facilities.

The following five tasks are performed in this model:

Data preparation
Rainfall characterization
Storage treatment balance
Overflow quality assessment
Receiving water response

The relative success of stormwater control alternatives may be checked at two major points in the simplified model. After the initial storage/treatment balance has been performed, the duration, volume, and frequency of overflows may be checked to determine the impact of a control alternative on the receiving water quality.

Process Cost/Benefit Models

Several cost/benefit models developed under the Rochester, New York CSO research and development project were utilized to evaluate the effectiveness of the various treatment process trains. Models were developed which relate an expected treatment efficiency to process and wastewater dependent variables.

As an example, the performance model for disinfection relates the effectiveness of the process to the disinfectant utilized, dose applied, velocity gradient in the disinfection chamber, influent bacterial level, wastewater BOD_5 and TKN levels, and the disinfection chamber contact time. To develop this disinfection performance model, a statistical analysis was conducted on the pilot plant data.

Using capital, operation and maintenance, material and supply, power, and chemical cost expressions, a cost performance model was developed. Using a cost optimization subroutine, the capital and operating costs associated with facilities having different disinfection requirements were developed, optimizing the operational and design parameters so as to minimize annual costs.

Similar cost optimization models are available for the analysis of a wide range of treatment processes. These tools were utilized in the cost/benefit analysis of the wet-weather treatment facilities in conjunction with application of the Runoff and Transport routines of SWMM Version II.

The reader is referred to Volume II of this Report for a complete description of the treatment process performance models used in the alternative analysis. A detailed description of the process cost models used in the CSO abatement alternative analysis is presented in Section 9.

SECTION 5

OVERFLOW MONITORING AND DRAINAGE BASIN CHARACTERIZATION

OVERFLOW MONITORING

General

The Rochester wastewater collection system is, for the most part, a combined system whereby sanitary sewage, industrial wastes, and stormwater are all collected and conveyed via the same conduits. The existing major trunk and main intercepting system serving the Rochester Pure Waters District is shown on Figure 1. The District is essentially divided by the Genesee River into western and eastern components. Overflows from the western portion discharge to the Genesee River, whereas, overflows from the eastern portion discharge to Irondequoit Bay. As stated in Section 2, the first important task in conducting a CSO pollution abatement program is identifying significant overflow volume and pollutant loadings from the existing sewer system.

Flow Monitoring

Thirteen major overflows to the receiving waters were monitored under this program as shown in Figure 8. Within each of these overflow discharge conduits an electronic flow measuring system was installed to determine the hydrograph and thus the quantity of combined sewer overflow discharged from the tributary drainage area. The primary flow measuring device was an ultrasonic level and/or velocity probe system manufactured by Badger Meter, Inc. In order to determine which ultrasonic system would be best suited to the existing conditions within the overflow conduits, intensive field evaluations were conducted.

Table 1 lists the location of the thirteen major combined sewer overflow monitoring installations and the installed equipment associated with each.

Besides measuring these thirteen overflows, four trunk sewer flow measurements were made. One was located at the point where the Genesee Valley Canal Sewer (GVCS) empties into the Clarissa Street Tunnel. The GVCS, for the most part, is a stone box sewer which is badly deteriorated. A replacement, the Genesee Valley Interceptor, is presently under construction. It

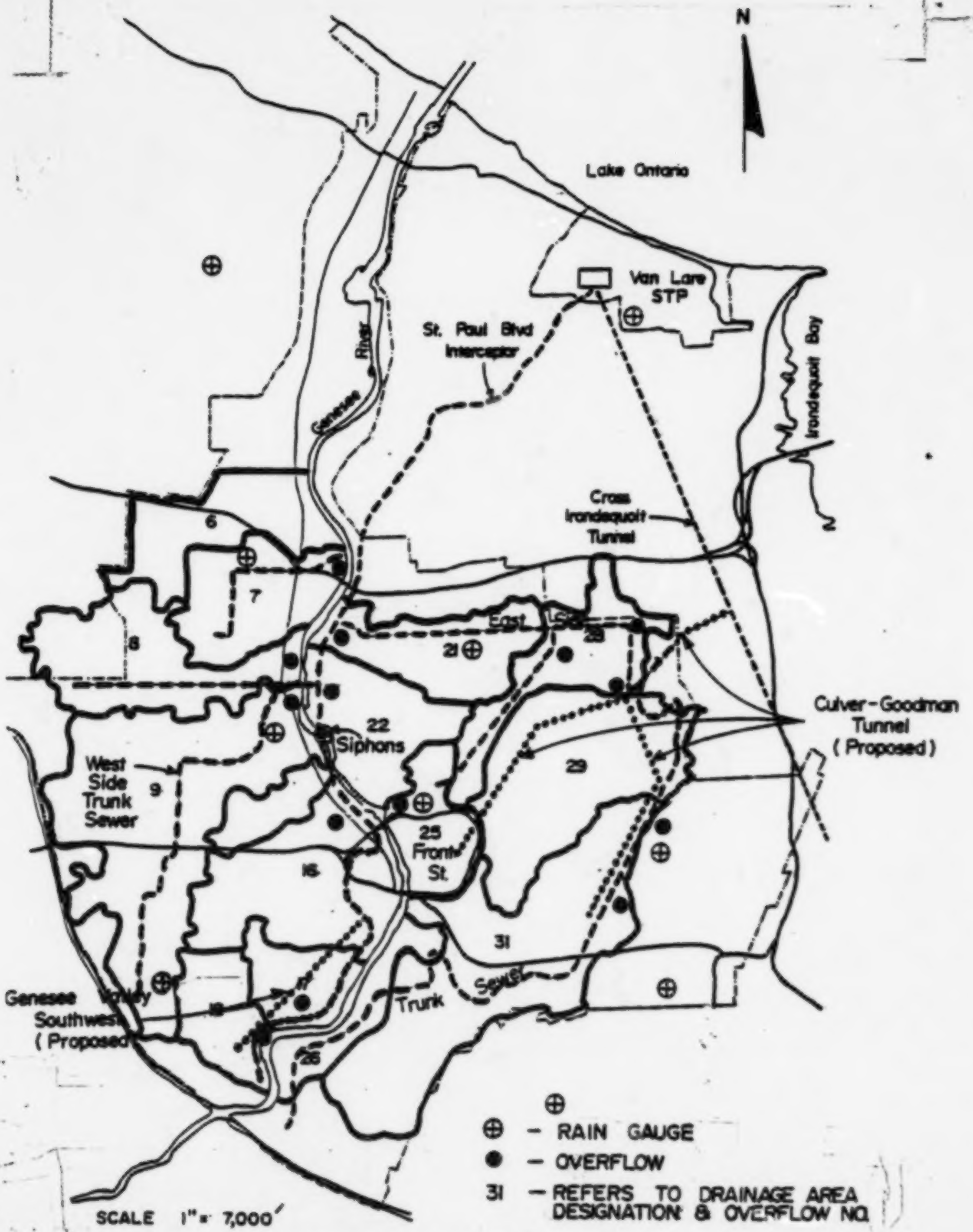

N
Lake Ontario
Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
6
7
8
East Side
21
28
22
Siphons
Culver-Goodman Tunnel (Proposed)
29
West Side Trunk Sewer
9
25 Front St.
16
31
Trunk Sewer
Genesee Valley Southwest (Proposed)
18
26
⊕ - RAIN GAUGE
● - OVERFLOW
31 - REFERS TO DRAINAGE AREA DESIGNATION & OVERFLOW NO.
SCALE 1" = 7,000'

FIGURE 8. Overflow Locations

TABLE 1. OVERFLOW MONITORING INSTALLATIONS

No.	Location	Ultrasonic Unit Installation Type
7	Maplewood Park	Head and Velocity
8	Lexington Avenue	Head
9	West Side Overflow	Head
16	Mill and Factory	Head
17	Plymouth and Railroad	Head
18	Brooks and Agnew Court	Head
21	Norton at Seth Green	Head
22	Carthage Avenue	Head
25	Central Avenue	Head
26	Court Street	Head
28	Norton Screenhouse	Head and Velocity
29	Densmore Bypass	Head and Velocity
31	Thomas Creek	Head

was felt that any actual flow records that could be obtained would be helpful in verifying the design capacity requirements for the new interceptor.

Two other interceptor flow measurements were made on the East Side Trunk Sewer (ESTS) at locations that provided information as to how the interceptor is operating at the same time that two regulators (tributary to Overflow No. 31) are diverting flow to overflow conduits.

The fourth interceptor flow measurement was made at the Van Lare STP. The existing influent flow measuring device is a venturi meter that has a maximum recording capacity of 200 mgd. At times of intense rainfall the flow to the Van Lare STP exceeds this amount, therefore, a meter with a higher range was necessary.

A typical head and velocity probe installation is depicted in Figure 9. The head probe installation was basically the same except for the deletion of the velocity probes. Each system generated a 4 to 20 milliamperes (ma) signal which was linearly proportional to the primary measurement (head and/or velocity). This signal was then converted at the overflow location to a tone signal and sent via telephone line to a central data acquisition station. The conversion and transmission of the tone signal was accomplished with telemetry instrumentation manufactured by Bristol Instruments, ACCO Bristol Division.

This tone signal was received and reconverted to a 4 to 20 ma output again with Bristol telemetry instrumentation. This output was then converted to a 0-10 v signal which was required for input into the Digital Equipment Corporation PDP8/E computer. The computer was programmed to translate the 0-10 v input into a flow value based on hydraulic curves established for each particular overflow site. The overall telemetry system is depicted in Figure 10. If the flow measurement involved head, a curve representing Manning's equation stored in the computer was used to compute the discharge (Q). However, if the monitoring installation involved both head and velocity, a curve reflecting Q=VA was stored where A, the flow area within the discharge conduit, was calculated from the water level sensing head probe.

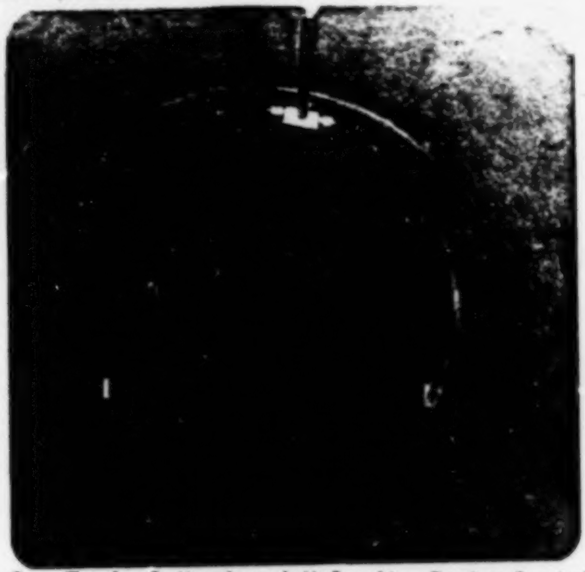


FIGURE 9. Typical Head and Velocity Probe Installation

The resultant flow values were printed and punched at five minute intervals. This interval was determined to be reasonable and could be changed by a simple command at the computer console. The punched paper tape was collected, translated, and the resultant overflow information stored on magnetic tape on the Xerox 560 computer located at the offices of O'Brien & Gere Engineers, Inc.

Sampling

Samplers were installed at each of the overflow locations in Figure 8. The samplers were refrigerated-sequential units that were equipped with 1/4 in. sampling hose. The pump and sampling apparatus were manufactured by Sigmamotor, Inc. and were very similar to their Model WM-5-24. However, modifications to the sampler package were necessary to meet the varying head conditions at each location.

For those overflow conduits that were more than 20 ft below grade, the sampling pump was installed in a waterproof box within 10 ft of the minimum overflow level. The pump then delivered the sample to an above grade location where the sample bottles were stored under refrigeration.

In order to coordinate the initiation of the sampler with the first measureable amount of overflow, a relay system was incorporated into the instrumentation already existing at each of the overflow locations. This

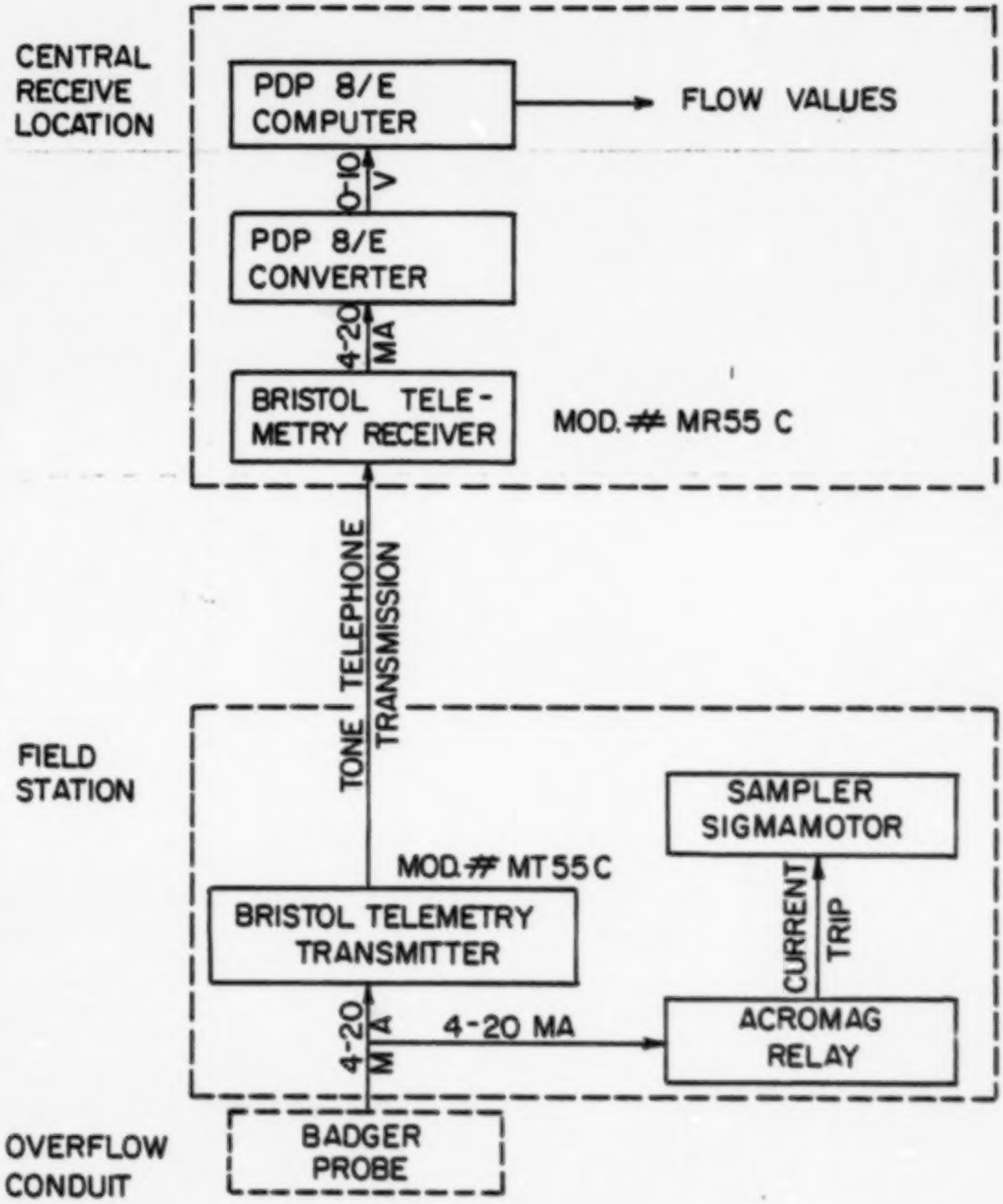

CENTRAL
RECEIVE
LOCATION
PDP 8/E
COMPUTER
FLOW VALUES
0-10 V
PDP 8/E
CONVERTER
4-20 MA
BRISTOL TELE-
METRY RECEIVER
MOD.# MR55 C
TONE TELEPHONE
TRANSMISSION
FIELD
STATION
SAMPLER
SIGMAMOTOR
MOD.# MT55C
BRISTOL TELEMETRY
TRANSMITTER
CURRENT
TRIP
4-20 MA
4-20 MA
ACROMAG
RELAY
OVERFLOW
CONDUIT
BADGER
PROBE

FIGURE 10. Overall Monitoring and Telemetry System

relay accepted the 4 to 20 ma signal from the Badger probe output and initiated the sampler when the 4 ma signal was increased to approximately 4.2 ma.

Samples were collected at approximately 15 min intervals unless the pumping distance dictated suction and purge times greater than 15 min. At the completion of an overflow event the samples were collected and transported to the laboratory for analyses. A portion of the analyses were performed by the Monroe County Division of Pure Waters laboratory and the remainder by O'Brien & Gere. Table 2 represents the schedule of analyses that were performed on the sequential and composite samples.

TABLE 2. OVERFLOW ANALYSIS SCHEDULE

Laboratory	Analyses	Type of Sample*
Monroe County Division of Pure Waters	BOD_5, Set. S, SS, TS, TVS, TDS, VDS, t. coliform, f. coliform, f. strep	S
O'Brien & Gere	Hg, O&G, Cl-Hydro.	C
	pH, COD, TOC, TKN, NH_3-N, Org. N, NO_3, NO_2, TIP, Cl, Fe, Cu, Zn, Pb, Cd, Cr, Ni	S

* S - Sequential; C - Composite

Rain Gauging

A system of 12 rain gauges was installed within the Rochester Pure Waters District. The location of each gauge was selected by dividing the District into 12 units of equal area. Within each of the units, a building of significant height (school, firehouse, pump station) with no observable influences from adjacent buildings or trees was selected for installation. The total system of gauges allowed an evaluation of differences of rainfall patterns and their possible effect on the intercepting sewer system. Figure 8 shows the location of each gauge.

All rain gauges were Fischer-Porter bucket-type instruments, Model No. 35B1558. These gauges recorded the amount of accumulative total rainfall at five-minute intervals on punch paper tape. The punch paper tape was collected approximately every two weeks, translated, and the information stored on magnetic tape.

The flow, analytical, and rainfall data that were monitored and recorded served two main purposes. First, the data were used directly in the calibration and verification process associated with the application of the SWMM. Secondly, much of the data was used for establishing valuable relationships between rainfall and overflow quantity and quality developed through the use of statistical procedures.

DRAINAGE AREA CHARACTERIZATION

Introduction

In the overall effort to develop an abatement and management program for the combined sewer overflows, the physical characteristics of the entire study area were defined. The collected data served several purposes: (1) it established a framework which allowed the successful utilization of the available network mathematical models, (2) it provided the necessary drainage area parameters for determining regression equations relating the pollutant loading of combined sewer discharges to tributary area characteristics,and (3) it provided the data base of surface characteristics that can be reasonably well correlated to that of other metropolitan areas.

Data Collection

A brief description of the scope of work outlined for the drainage area surveys follows:

1. Each drainage basin was divided into a maximum of sixty subcatchments ranging in size from ten to fifty acres. Division was made on the basis of land slope and zoning classifications such as residential, commercial, industrial, or open.

2. Each drainage area subcatchment was characterized by determining the average ground slope, percent of the area that is pervious and impervious, total area, average drainage width, total lenth of gutters, total number of catchbasins and stored volume of each, the BOD_5 contained in the volume of stored water in those catchbasins, and any available surface storage capacities.

3. The estimation of dry-weather flow (DWF) for each subcatchment was made by collecting statistics as to the number of dwelling units, average number of people per unit, market value of average unit, average family income, percent of units with garbage grinders, and information as to industrial process flows.

4. Other information required to complete the characterization of the drainage basin included total population, street cleaning frequency, average number of sweeper passes, and location of inlet manholes.

5. Information was obtained to characterize the sewer network including length of sewer conduits, conduit slope, type, size and roughness, and manhole invert and ground elevations.

The drainage basins identified in Figure 8 were initially defined using the District's sewer maps in conjunction with previous reports (4, 5) and selective field investigations. In addition, the level of detail for the drainage basin characterization resulted from initial interpretations of the data base requirements of the mathematical network models. The level of effort required for drainage basin characterization is largely dependent on the purpose of the study and should be identified in the methodology of approach. Tables 3 and 4 summarize the field determined drainage basin characteristics conducted under this program. These data were used in conjunction with the mathematical network modeling effort.

TABLE 3. SUMMARY OF DRAINAGE AREA CHARACTERISTICS

Drainage Area No.	Area ac	Land Use - ac SFR*	MFR*	Commercial	Industrial	Open	Ave. Land Slope	% Imp.*
6 (separate)	569	247	16	24	130	152	0.0074	53.2
7	729	610	7	53	18	41	0.0118	48.2
8	988	339	22	32	465	130	0.0066	46.4
9	2682	1407	0	109	994	172	0.0060	49.2
16	826	348	10	318	50	100	0.0059	66.1
17	235	196	9	6	0	24	0.0067	41.6
18	541	507	3	19	0	12	0.0073	41.6
21	810	644	0	72	55	39	0.0073	44.9
22	569	434	0	38	27	70	0.0070	52.9
25	348	0	91	257	0	0	0.0080	80.3
26	393	118	40	175	20	40	0.0100	43.3
28	778	506	78	78	38	78	0.0100	46.6
29	1353	910	83	140	70	150	0.0100	54.4
31	1671	820	155	320	240	136	0.0100	49.9
Tryon Park (separate)	1720	1118	344	86	86	86	0.0150	40.0
Charlotte	1390	695	139	278	139	139	0.0150	35.0

*Note:
SFR = single family residential
MFR = multi-family residential
% Imp. = percent imperviousness

TABLE 4. SUMMARY OF DRAINAGE AREA CHARACTERISTICS

Drainage Area No.	Dwelling Units	Population	Catchbasins	Gutter Length-100 Ft.	No. of Conduits & Subcatchments in Original SWMM Network	
6	1300	3948	527	1015	139	29
7	4653	18476	911	2022	133	60
8	2016	5522	574	1353	72	32
9	9919	27296	2357	6481	136	60
16	4397	13687	(826)	(2800)	32	28
17	1979	6408	335	743	61	28
18	3965	11980	736	1730	114	43
21	6333	17540	1339	2895	128	54
22	4596	14332	-	-	95	52
25	0	(12700)	-	-	24	11
26	(3500)	(16700)	-	-		
28	5287	16467	-	-	118	40
29	(10700)	29700	-	-	256	83
31	(9200)	25500	-	-	122	48
Tryon Park	-	-	-	-	-	-
Charlotte	-	-	-	-	-	-

NOTE: Values in ( ) indicate extrapolated or assumed data based on the other drainage areas. No value indicates parameter not used as SWMM input.

SECTION 6

NETWORK MODELING

CONFIGURATION AND TYPE OF MODELS USED

Introduction

Until recently, the most common methods for analyzing and designing storm and combined sewer systems have been based on the rational formula and various steady-state flow equations for sewer network analyses. Mathematical models have been developed in response to the recognized limitations of previous methods for analyzing highly complex systems under dynamic flow conditions. The models consider a number of factors:

(1) Spatial nonuniformity of rainfall
(2) Randomness of rainfall events
(3) Time variable runoff
(4) Spatial and temporal variations in dry-weather flows
(5) Overland surface runoff travel times
(6) Sewer conduit flow routing
(7) Operation of diversion structures and storage facilities
(8) Pollutant quality characteristics of surface runoff and dry-weather flow.

Minimum requirements of the selected network models include the capability to consider several representative rainfall events, compute runoff from several subcatchments or drainage basins, and the ability to route flows in a branched conveyance network.

The two models selected for demonstration and use in the evaluation of various combined sewer overflow abatement alternatives proposed under this program were the SWMM (11) and the Simplified Stormwater Model (7). The Simplified Stormwater Model was developed by Metcalf & Eddy, Inc., and demonstrated by O'Brien & Gere Engineers, Inc. under this program. Each model was selected for a specific purpose. Preliminary screening of various abatement alternatives was accomplished with the use of the Simplified Stormwater Model, whereas, the SWMM was used to evaluate specific alternatives under the application of varying design storms.

Stormwater Management Model (SWMM)

In 1971, a joint venture of Water Resources Engineers, Inc., Metcalf and Eddy, Inc., and the University of Florida developed a mathematical model capable of simulating urban stormwater runoff and combined sewer overflows. The model accepts rainfall and drainage basin characteristics as inputs and calculates the quantity and quality of stormwater and combined sewer overflow.

Basically, the model consists of four components: runoff, transport, storage, and receiving water. The runoff component computes direct runoff for a given rainfall event for each subcatchment using a kinematic wave formulation with Manning's equation.

The generated runoff then serves as input to the sewer system network. The transport component routes the flow through the sewer system using a modified kinematic flow approximation. This analysis combines Manning's equation and the conservation of mass. The storage component modifies the sewer flow within the system at selected points where such facilities have been provided. Finally, the receiving water component computes the effects of the stormwater and combined sewer overflow discharges on the receiving waters.

More specifically, the basic elements of the SWMM program are built on the following major routines (the capitalized, underlined names represent computer language subroutines as identified in the EPA documentation of the SWMM).

(1) Input Sources

RUNOFF generates overland surface runoff based on input rainfall hyetographs and surface characteristics such as imperviousness, land use, and infiltration capacity. Surface quality characteristics of urban runoff are also generated in this routine and are based on factors such as antecedent rainfall conditions, street cleaning practices, and land use.

FILTH generates the quantity of sanitary and industrial flow based upon such factors as land use, population density, and process flows.

INFIL generates infiltration into the modeled sewer system network based on groundwater and sewer conduit conditions.

(2) Central Processing

TRANS combines and routes the generated surface runoff quantity and quality values through the sewer system. However, due to possible surcharge and backwater conditions existing in the Rochester sewer system, a different version of this particular routine had to be used to better represent actual flow conditions in the sewers.

For the network modeling of the Rochester Pure Waters District sewer system with SWMM, several modifications had to be made to the SWMM program. First, the Water Resources Engineers, Inc. (WRE) modified version of transport block had to be used due to possible surcharge conditions within the sewer system (12). The primary differences between the original SWMM transport and the WRE version of the transport block can be summarized as follows:

(a) Conceptualization of the transport system with the WRE model uses a more refined mathematical representation of the sewer system totally unlike the original SWMM. This allows for more accurate modeling and analysis of the actual system.

(b) The basic flow equation in the WRE model includes the inertial terms of the Saint-Venant equation in the solution, whereas, the original SWMM is based on a kinematic wave approach. The inclusion of these other terms allows for a better dynamic mathematical simulation of the sewer system with existing surcharge and backwater conditions.

(c) The WRE model includes the necessary programming to simulate sewer surcharge, looped systems, weirs, and other related features that were unavailable with the original SWMM.

It should be noted that the SWMM computer program that was originally developed in 1971 has been undergoing constant review and improvement. Many routines that were initially unavailable are now part of, or are included as, options to the SWMM program. The specific SWMM version used for the Rochester system analyses is Version II dated March 1975. More recent versions also include as an option the Water Resources Engineers transport routine, frequently referred to as WRE Transport, extended transport, or simply EXTRAN. At the time this study was being conducted this particular option was not available as part of SWMM Version II.

It should also be noted that in the SWMM Version II runoff subroutines, the dust and dirt pollutant build-up factors are considered to be concentrated over the paved street areas and not uniformly over the entire drainage area.

The quality routing section of the WRE model in which individual pollutants are transported by advection in each modeled conduit, theoretically provides a better representation of actual processes than the original SWMM (12). In the present link-node version it is assumed that all transport of constituents occurs by advection in the system conduits, while dilution of pollutant concentration occurs at the system nodes. This means that only those pipes which enter a node from upstream can advect pollutant material into the node. Thus, the pollutant concentration at a given node is unaffected by quality events downstream from the node. This is valid for gradually varied flow in a conduit system assumption.

Although the original storage/treatment routine of the SWMM was intended to be used to evaluate unit treatment operation and capital intensive storage systems, many of the mathematical formulations related to process definition and efficiencies are internal to the model and are therefore not user controlled. Pilot plant evaluations conducted as part of the original scope of work for this study indicated that many of the inherent model equations were not representative of the response of a number of unit processes to the combined wastewaters characteristic to the Rochester system. It became more feasible to develop empirical relationships relating flow and treatment efficiency and system costs from these pilot plant studies than to attempt to modify the equations that are presently incorporated in the model.

The SWMM incorporates a receiving water simulation routine that can be used to predict water quality impairment from combined sewer overflow discharges. Since little prior work had been done with this portion of the SWMM and because of the availability of a verified receiving water model, the LIMNOSS model (which is a modification of the EPA steady-state AUTOQUAL model) was adopted for use (13). The LIMNOSS model is a one-dimensional, second order, finite-difference simulation model. Projections of water quality effects on the Genesee River from the present system and various abatement alternatives were made using this model.

Machine requirements for the entire SWMM involves a high speed computer with approximately 90K words of storage. Available core storage on the Xerox 560 located at the offices of O'Brien & Gere Engineers is approximately 30K. Through the use of offline disc storage and retrieval, the deletion of some routines in the SWMM, and the extensive editing of the program, the SWMM program was adopted to be executed on the Xerox 560. The system dictated program modifications involved the addition of a paging algorithm allowing the large arrays associated with the original SWMM to be compatible with the available storage.

Simplified Stormwater Model (SSM)

A simplified stormwater management model has been developed by Metcalf & Eddy, Inc. to provide an inexpensive, flexible tool for planning and preliminary sizing of stormwater and combined sewer overflow abatement facilities (7). It successfully introduces time and probability, through the use of long-term historical rainfall data, to the estimation of the frequency, duration, and quantities of stormwater during rainfall events.

The use and application of the simplified model involves five main tasks:

(1) Data preparation
(2) Rainfall characterization
(3) Storage/treatment balance
(4) Overflow quality assessment
(5) Receiving water response

Data preparation consists of synthesizing a schematic of the sewer system network to be evaluated and subsequent collection of data on both overflow quantity and quality. This task provides the necessary information for input to Tasks 3 and 4. The rainfall characterization is easily accomplished through a small computer program introduced as part of the model and can be used to evaluate various design storms and to produce a historical perspective on the storm events and their impact on the sewer system. The storage/treatment portion of the model can be used repeatedly to analyze various combinations of storage capacities and interceptor conveyance capacities to determine the optimum conditions required to minimize overflow.

Strictly speaking, the overflow quality assessment and the receiving water response (Tasks 4 and 5) are not an integral part of the SSM. Therefore, modeling of the Rochester sewer system with the SSM involved only tasks 1 through 3. Evaluation of overflow quality and receiving water response were made using statistical procedures and alternate models that are not integral to the SSM.

The main purpose of the SSM was to provide a preliminary evaluation of storage and treatment facilities necessary for combined sewer overflow control. The conceptual basis for Task 3, storage/treatment, is shown schematically in Figure 11.

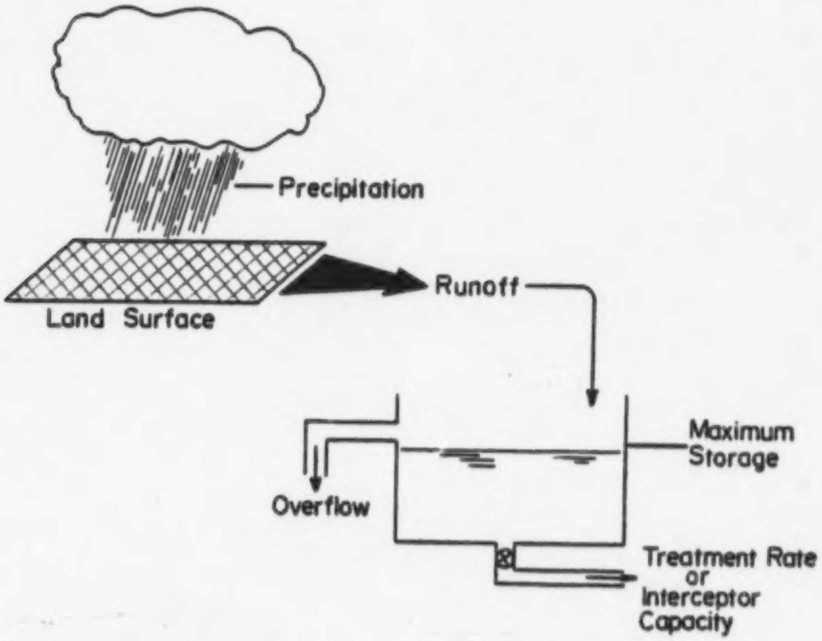


Figure 11. Concept of Storage/Treatment-SSM Analysis

The computer program can accept rainfall from either a long-term historical record or from an individual storm event. Rainfall is then converted into runoff using a gross runoff coefficient. The generated runoff is stored in a defined storage volume (which can be zero) which is drained at a specified treatment rate. The defined treatment rate could be represented by an actual treatment facility or by available interceptor capacity. When the runoff rate exceeds the storage capacity and the treatment rate, an overflow occurs.

The major data required to run the storage/treatment portion of the model involve the catchment area, the gross runoff coefficient of the overall drainage area, the in-system storage capacity, and the treatment rate or interceptor capacity. It must be noted that the mathematical model does not involve a complex dynamic hydraulic mathematical simulation. It does not route generated runoff volumes through a sewer network according to established fluid mechanic flow equations, but rather accounts for rainfall and subsequent runoff on a steady state mass balance - volumetric basis.

The SSM output is presented in the form of a record of the time and volumes of rainfall, runoff, and overflow, and a monthly summary of these parameters. Minor programming modifications made to the basic SSM for this study include a statistical analysis of the generated output, which provides the user with the average, standard deviation, confidence limits, and other statistical information associated with the output parameters.

Water Quality Models

Any combined sewer overflow abatement program has, as an ultimate objective, the establishment of acceptable receiving water quality conditions and the preservation of the best usage of the aquatic resources. To achieve this goal, it is necessary to project the effects of both present combined sewer discharges to receiving water bodies as well as the discharges resulting from the implementation of various abatement alternatives. As previously mentioned, the output of the LIMNOSS receiving model generated under a concurrent program (13) was used to provide the necessary background receiving water definition as well as the required water quality projections under varying loading conditions for the abatement alternative analyses.

Three LIMNOSS receiving water quality models were formulated for the purpose of evaluating the impact of existing and projected combined sewer overflow pollutional loads to the Genesee River and to the Rochester Embayment of Lake Ontario. The models include a steady-state fecal coliform and assimilation capacity simulation program for the impacted region of the Genesee River and a steady-state fecal coliform model for the Rochester Embayment. The input required for preparation of the above defined models was acquired through dry and wet-weather sampling. The structure of the model is based on the simple concepts of continuity, including terms for advection, dispersion, reaction, and point or non-point sources or sinks. The model can calculate the steady state spatial distribution of any water quality parameter whose kinetic behavior can be described by zero or first order kinetic formulations or described as conservative. In the case of the

Genesee River, the parameters modeled included chloride, fecal coliform, carbonaceous oxygen demand, nitrogenous oxygen demand, and dissolved oxygen.

The model requires detailed data input of physical parameters (width, depth, length, etc.), loads, flows, reaction coefficients, and dispersion coefficients. The model assigns specific values to each model segment, sets up a water quality matrix according to the mathematical formulation as described above, and then solves the system of differential equations using an algebraic inversion of the equation matrix. The model has been carefully verified for theoretical and numerical accuracy using a number of test cases where direct analytical solutions are available for comparison (13).

ASSUMPTIONS IN MODELING

Runoff Projections

An intensive effort was made in the collection of field data that was established as necessary for drainage basin characterization for subsequent use in mathematical modeling utilizing the SWMM. Much of the collected data pertained to input information for the runoff block of the SWMM. Initially, the various drainage basins were broken down or discretized into a large number of smaller catchment areas each with its own input parameter characteristics, such as percent imperviousness, area, overland flow, detention storage and resistance factors, the number of catchbasins and the length of curbing. Sensitivity analysis conducted on the runoff block of the SWMM indicated that the model was quite insensitive to many of the input parameters in relation to the resultant runoff hydrograph. These input parameters include ground slope, resistance factors for impervious and pervious areas, and width of subcatchment. Parameters which significantly changed the resulting runoff hydrograph included area, percent imperviousness, retention storage and infiltration rates. In addition, the analysis showed that for specific drainage basins the resultant runoff hydrograph obtained from either a coarse or a detailed discretized area was essentially the same. The coarsely discretized area required much less computer execution time thereby resulting in significant savings. A coarse network further allowed for less manhours in computer set-up time. The average savings was estimated to be by a factor of ten considering both computer costs and labor.

It must be noted, however, that the runoff from a particular drainage area was not monitored during this study. Only the overflow from the sewer system was measured. The runoff projections from the SWMM were refined based on the comparison of projected to measured overflow volumes. Calibration of the model is also described in this section.

Application of the runoff block of the SWMM involved several assumptions. The default values (values that the model assumes if not user supplied) for several catchment area parameters were considered applicable for the Rochester area. The specific parameters involved were the surface runoff resistance factors, surface depression storage, and soil infiltration rates. For most of the storm events that were considered, the default values were considered sufficiently accurate and thus were utilized.

In addition to the volume of runoff, the runoff block of the SWMM accounts for the quality of surface runoff. Initially, the model estimates the amount of pollutants accumulated on the surface prior to rainfall. Then, accounting for rainfall intensity, land use, and soil erodibility, the model projects the quality of runoff. The output is in the form of a loadograph or the time varying pollutant concentration profile. Input parameters include the number of antecedent dry days, street cleaning frequency and number of passes, number of catchbasins and their stored volume and BOD_5 content, erosion related soil factors, and the length of gutters within the catchment area. For this study erosion was not considered a factor due to the large percentage of paved and rooftop areas. In addition, on the basis of small samplings, the number of catchbasins and the total length of gutters on a per acre basis were assumed to be essentially uniform throughout the drainage basins. The problems associated with the runoff block quality subroutine will also be discussed in this section.

Overflow Projections

It was determined that the most important factor in applying the WRE Transport routine is the selection of the integration step. Due to the mathematical representation of the physical system, the optimal length of the integration step is related to the length and diameter of the sewer element modeled. The length of time-step required is directly proportional to the length and inversely proportional to the diameter of the sewer conduit. Thus, short, large diameter conduits require very short time steps to maintain mathematical model stability.

As previously mentioned, coarse discretization of the drainage areas allowed for the development of a coarse yet reliable representation of the sewer network based on comparisons between model predicted values and monitored quantities. This resulted in relatively long lengths of conduits between manholes. Even with a coarse network, the relative magnitudes of length and diameter of conduits necessitated the selection of a small integration step. For most model runs a five second time step was considered sufficient. Any additional accuracy obtained from runs with a shorter time step was not considered justifiable.

Another important aspect of applying the WRE Transport routine was the manner in which the actual sewer system was mathematically represented for model input. By including all elements of the sewer system such as manholes, conduits, weirs, orifices, pump stations, and other appurtenances, accurate model overflow projections would result but very long computer simulation times would be incurred. Consequently, a coarse representation of the network was selected and modeled. It was assumed that the average slope of a sewer conduit over a long distance would be equivalent to modeling many intermediate lengths between manholes. In addition, only pipes larger than 12 in. in diameter were modeled. No pumps were included and only those control or regulator structures which diverted combined sewage to the receiving waters were modeled. Furthermore, although usually there are varying shaped sewer conduits within a large urban area, almost all the conduits for this study were modeled as equivalent circular pipes because of the limited number of shapes analyzed by the WRE Transport routine. If the main concern is the overflow hydrograph, then a network composed of hydraulically equivalent conduits would be sufficient.

Inasmuch as the modeled network is a combined system, most of the flows associated with overflow events result from rainfall induced runoff. Dry-weather flow and infiltration during a storm event played a minor role. In fact, infiltration during a storm event was considered negligible and therefore not modeled. Regarding dry-weather flow, although there are many model input parameters that require definition, in general, model default values were considered sufficient. The only concern was to match total dry-weather flow in the modeled system to that being recorded at the Van Lare STP.

LIMITATIONS OF THE AVAILABLE SOFTWARE

Runoff

In the overall rainfall-runoff SWMM simulation,all wastewater flows and pollutants, except for the contribution of dry-weather flow and groundwater infiltration, are generated within the runoff routine of the model. Therefore, accurate projections of combined sewer overflow hydrographs and pollutographs depend to a large extent on the accuracy of the runoff block-generated hydrographs and pollutographs. Actual runoff hydrographs from individual drainage areas, however, were not monitored. Only the system overflow from the area was monitored during a storm event. For a mass balance check at each regulator or diversion structure, the incoming flow originating from surface runoff entering the sewer system, the wastewater flow in the trunk or intercepting sewer, and the overflow must be known. From the monitoring data obtained on the overflows and limited interceptor measurements the accuracy of the model runoff projections could not be assessed directly. From other runoff calculations performed and from the fact that the overflow projections generally corresponded to those monitored it is believed that the calibrated SWMM runoff routine computes the surface runoff with sufficient accuracy, given a specific rainfall event.

Combined sewer overflow wastewater quality was also modeled as part of the overall overflow monitoring effort. The SWMM incorporates two separate routes for the calculation of overflow quality characteristics. The surface washoff contribution is calculated as part of the runoff routine of the model. The impact of pollutants contributed from the dry-weather flow and industrial discharges is accounted for within the transport routine of the model. An extensive effort was undertaken to correlate model overflow quality projections to those monitored. Because it is difficult to separate the storm-generated pollutants from the dry-weather flow contribution, the actual accuracy of the runoff quality portion of the model could not be ascertained. It is believed, however, that various parameters such as the dust and dirt accumulation rate and the associated fraction of BOD_5 and SS accounted for within the runoff routine need refinement.

It must be further noted that, according to the SWMM runoff block, all urban contaminants are assumed to derive from street surfaces only; they are removed by runoff up to the amount of pollutant remaining on the ground.

WRE Transport

This portion of the SWMM is very complex and a major effort is required to implement it. Limited documentation at the time of the study makes the model very difficult for the user to implement without exhaustive efforts in the interpretation of the theoretical base and the definition of the input data set requirements.

Improvements are also needed in the model output format. The arrangement of several tables makes it difficult for the user to interpret the overflow projections. In addition, complete hydrographs and pollutographs may be obtained for only twenty selected locations within the sewer system. If additional points need to be evaluated, repeated model runs of the same data are required to obtain sufficient information.

Specific limitations of this routine include:

(1) Inability to model various shaped sewer conduits other than the six provided for within the computer program,

(2) Inability to account for water loss from the system as the result of surface flooding from surcharged manholes,

(3) Model is event-specific and does not include provisions for antecedent moisture conditions or water quality accounting between rainfall events, and

(4) Inability to accurately define the necessary time interval or integration step necessary for mathematical simulation based on the defined differential equations for flow definition.

Most of the above limitations have been recognized as serious problems and model improvements are currently in progress, but no reasonable explanation or interpretation has been found for the integration step problem.

In general, overflow projections provided by the SWMM appear to be correlated reasonably well with actual monitored field data relative to overflow discharge rates and volumes. The accuracy of wastewater quality predictions may be expected, however, to have only order-of-magnitude accuracy. Considerable model improvement is required in terms of its ability to simulate overflow quality before the model can be used with confidence for such projections.

Storage and Treatment

A serious limitation of the model was encountered during application of the treatment routine. The model does not accurately predict BOD_5 and TSS removal efficiencies of various treatment process devices.

Most of the programming for this portion of the SWMM does not appear consistent with the results of the pilot plant studies which evaluated numerous process trains. In order to expedite development of water quality projections, it was established that entirely new equations had to be developed to accurately analyze treatment of combined sewer overflows. These relationships were developed based on the regression analysis of performance data as described in Volume II of this Report.

Receiving Water

Although the theoretical and numerical accuracy of the previously described steady state water quality models that were used in the overall combined sewer overflow abatement program have been thoroughly verified, their suitability for describing the impact of transient loading phenomenon may still be questioned. A steady state water quality framework can be used to approximate the magnitude of the water quality impact; however, the framework is quite limited in simulating the real time response of the Genesee River to stormwater discharges. The experienced time variations in the discharge loadings and the river flow simply cannot be accurately described by a steady state model. The use of this model was intended only to provide a relative comparison for the river response to stormwater discharges, and to bracket the expected resulting water quality.

The one-dimensional assumption of the model also imposes some limitations on modeling the Genesee River. However, these efforts are not considered overly significant because intrusion from Lake Ontario and vertical stratification are not pronounced. In general, the model framework is considered satisfactory to describe dry-weather conditions in the Genesee River if, and only if, perturbations resulting from varying loads and photosynthetic activity are not considered important. Its suitability to describe wet-weather conditions in the river is adequate only for assisting in qualitative evaluations and to bracket the projected water quality. Establishing accurate numerical predictions of pollutant concentrations is in general difficult because of transient conditions induced by stormwater and combined sewer discharges.

MODEL CALIBRATION AND VERIFICATION

Overflow Volume

In order to expedite verification and model utilization in abatement analysis, the SWMM was calibrated and verified for three drainage areas and then applied to the remaining areas. The areas selected for calibration and verification were Maplewood Park, Carthage, and Thomas Creek which represent drainage areas numbers 7, 22, and 31, respectively. The decision to use these areas was based on the extensive amount of measured flow and quality data obtained from the overflow monitoring program for these sites. In addition, Carthage was selected because its overflow exhibited extremely high pollutant mass loading rates relative to the volume discharged. Furthermore, the three drainage areas exhibited the range of land use categories characteristic of the entire study area.

The storms involved in the SWMM calibration process were of both varying intensities and durations. In general, the rainfall data were obtained from the rain gauges associated with the drainage areas as part of the total overflow monitoring program. The exception is for Carthage where the required rainfall data were obtained from the U.S. Weather Bureau at the Monroe County Airport in addition to rain gauges located in adjacent areas.

It must be remembered that under the monitoring program, only the resulting overflow discharge rate was measured, not the runoff from a particular drainage area. As such, model predicted overflow hydrographs were compared to actual monitored data. Therefore, the WRE transport routine was used for calibration and verification. It is the transport routine of the SWMM that generates the overflow quantities based on the sewer network characteristics. The runoff routine estimates the rainfall-runoff relationship. This was indirectly verified.

For each of the three drainage areas selected for verification and calibration, three storms were identified for modeling purposes. The three selected storm events exhibited the range of rainfall intensities, durations, and volumes likely to be encountered during an average year. From the rain gauge data, rainfall hyetographs were constructed for model input. Table 5 shows the selected storms and their important characteristics for each of the three drainage areas.

These hyetographs provided the required rainfall input to the SWMM runoff routine which in turn generated runoff hydrographs for each drainage area. From the SWMM transport routine, which accepted the runoff hydrographs and added in domestic and industrial wastewater discharges, the combined sewer overflow volumes from each drainage area were generated.

TABLE 5. RAINFALL CHARACTERISTICS

Drainage Area No.	Storm	Rain-Gauge No.	Peak Intensity (in./hr)	Duration (hr)	Total Volume (in.)
7	10/9/75	3	0.12	4.33	0.40
7	9/11/75	3	0.24	2.92	0.40
7	9/20/75	3	0.20	1.00	0.20
22	8/29/75	5	0.60	7.92	0.80
22	8/24/75	Airport	0.42	8.00	0.71
22	9/11/75	5	2.40	3.17	0.80
31	6/19/75	2	0.60	3.68	0.50
31	6/ 5/75	2	1.20	1.36	0.60
31	6/12/75	2	1.20	2.56	0.80

Note: 1 in. = 2.54 cm

The most significant characteristics of the predicted overflow hydrographs for calibration and verification purposes were the peak flowrate, the total overflow volume, and the general shape of the hydrograph. Figures 12 through 20 show the predicted and measured overflow hydrographs, plotted as cfs against clock time, for each of the three selected drainage areas for each of the three storms. The first figure presented for each drainage area represents calibration runs while the other two represent verification runs. As indicated, Thomas Creek exhibited the best correlation between predicted and measured overflow hydrographs. The predicted total volume for each storm was within 10 percent of the measured value while the predicted peak flowrate was within 2 percent of the measured value. The correlation for Carthage also was very good, although the results for the storm of 9/11/75 showed a variation in peak flowrate and time frame between measured and predicted volumes. The predicted total volume for each storm was within 16 percent of the measured quantity while predicted peak flowrates were within 34 percent of the measured values. The largest discrepancies between the SWMM predicted volumes and measured overflow quantities occurred for the Maplewood Park area. The calibration and verification show very good correlation between model projections and measured field data. Table 6 shows the total volumes and peak flowrates for the predicted and measured overflows by site and storm.

The SWMM was calibrated and verified for the purpose of analyzing the effectivensss of various abatement alternatives in reducing the quantity of combined sewer overflow being discharged from the collection system. For this type of analysis, a simplified sewer network of the entire Rochester Pure Waters District was characterized for model evaluation. Figure 21 shows the conduits and nodes representing the entire combined sewer system. Trunk sewers from the individual drainage areas discharge to the main interceptor at rates governed by system regulators. It is important to note that the principal combined sewer overflows occur on trunk sewer systems prior to discharge to the interceptor. There are only two overflow relief points on the main interceptor system leading to the Van Lare STP. One is at Front Street and the other is at the siphon under the Genesee River where the St. Paul Boulevard Interceptor crosses from the west to the east side. These two locations are designated as nodes 250 and 251 in Figure 21.

As with any model simulation of actual conditions, it is impossible to exactly represent the sewer network as it occurs in the field. The simplified network represented by Figure 21 is hydraulically equivalent to the actual sewer network consisting of hundreds of manholes (nodes) and pipes (conduits). That is, the conduits represented in Figure 21 have similar flow characteristics to the actual conduits on an average overall basis. Because of the good correlation between predicted and measured overflow hydrographs for the three selected drainage areas, application of the same technique to the remaining drainage areas was justified. It is for this reason that the total overflow volumes from the entire system are believed to be within 20 percent of actual.

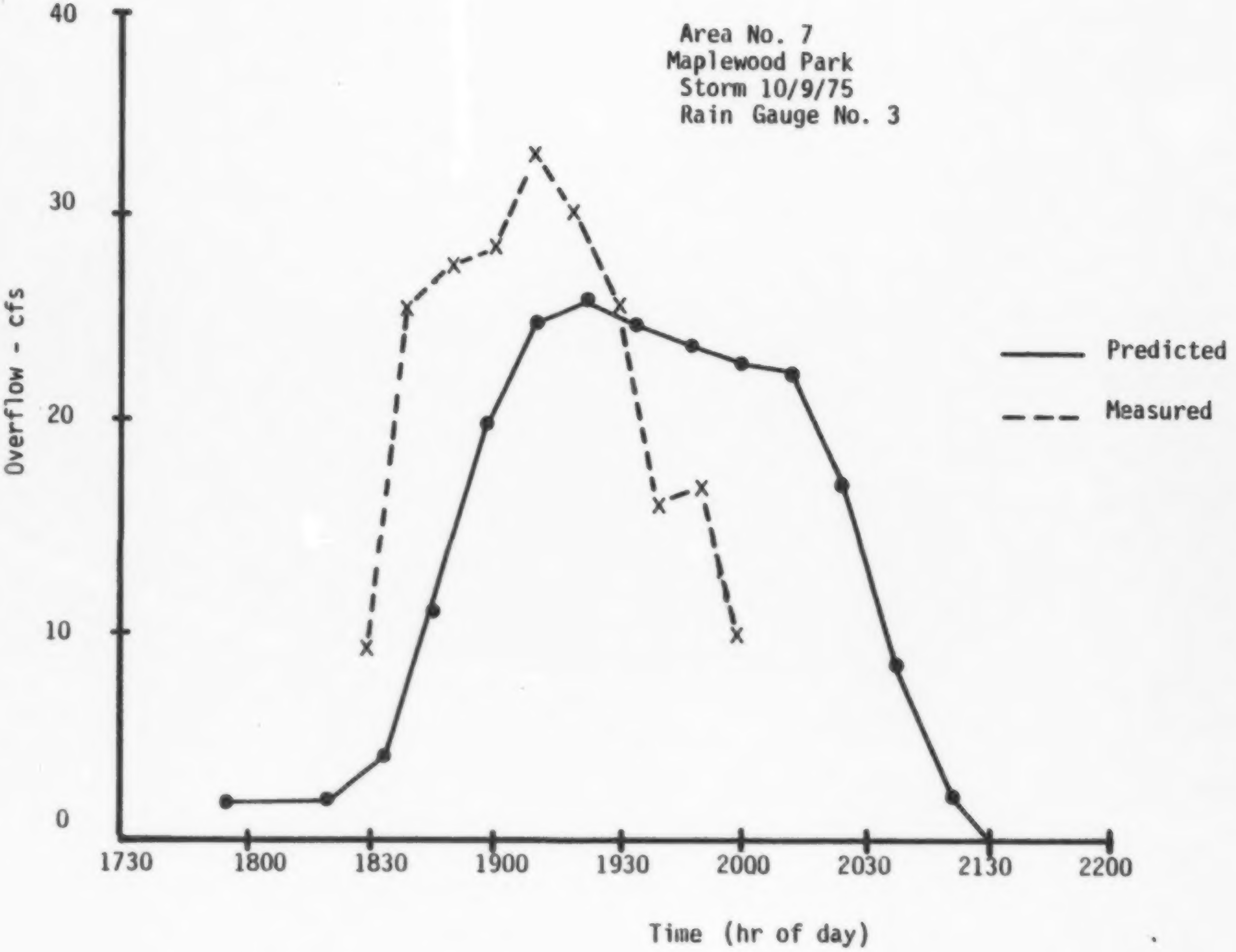


FIGURE 12. Calibration Overflow Hydrograph for Maplewood Park 10/9/75

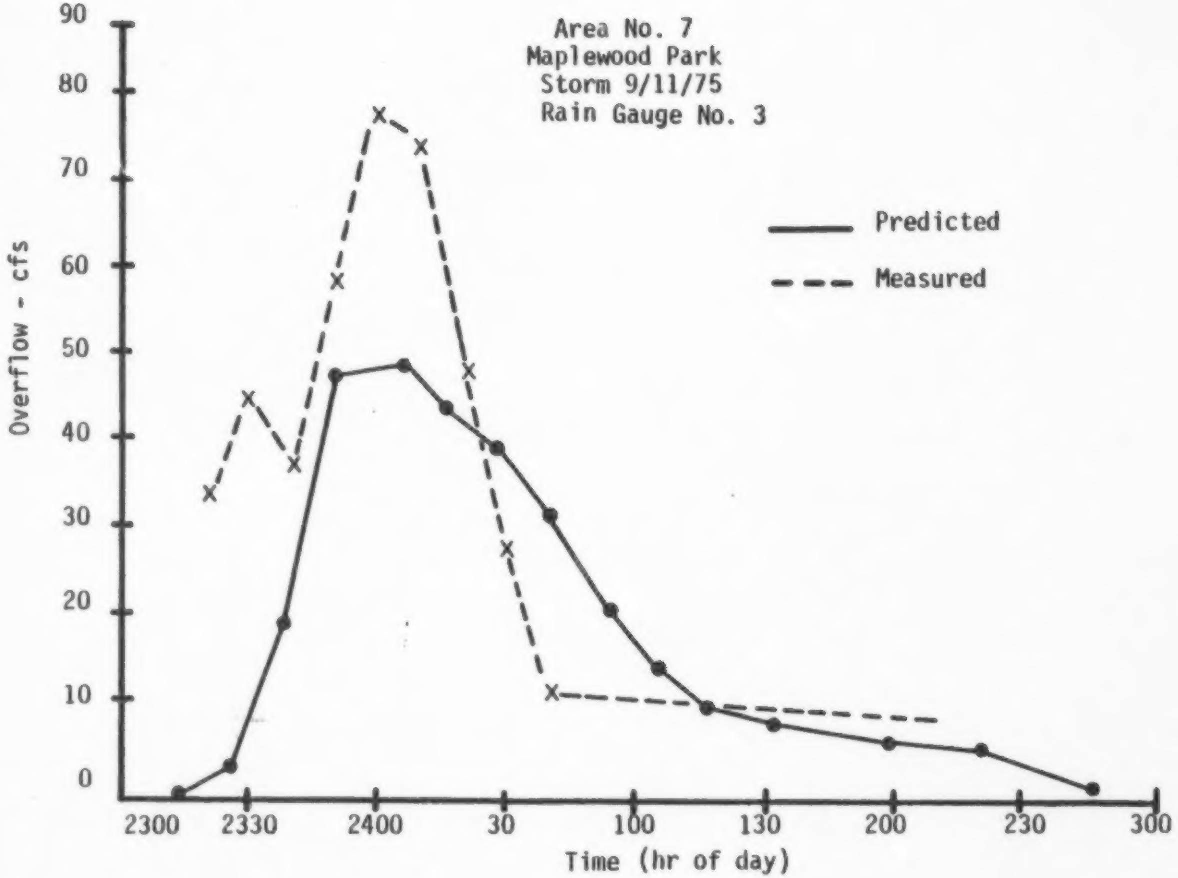


FIGURE 13. Verification Overflow Hydrograph for Maplewood Park 9/11/75

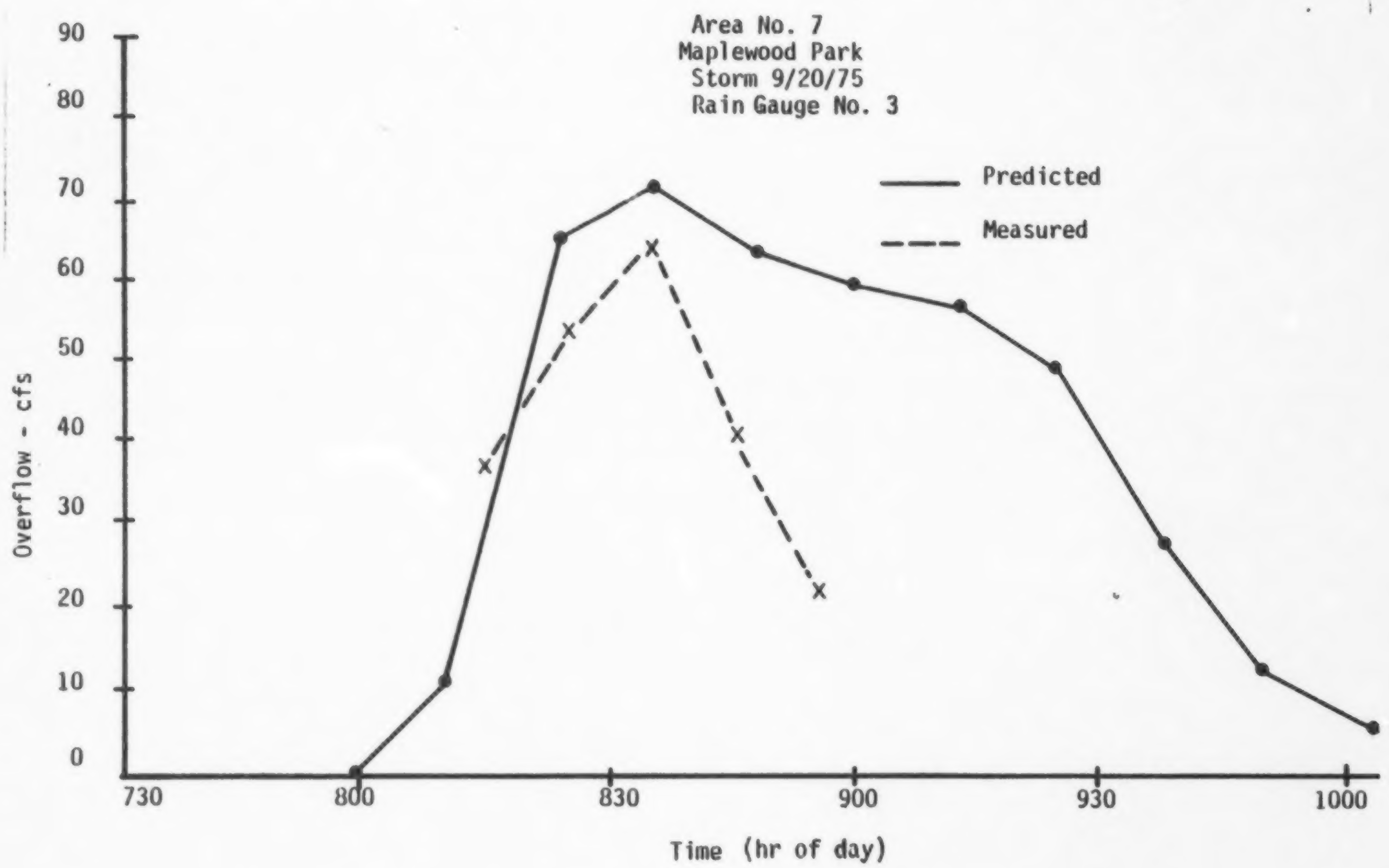


FIGURE 14. Verification Overflow Hydrograph for Maplewood Park 9/20/75

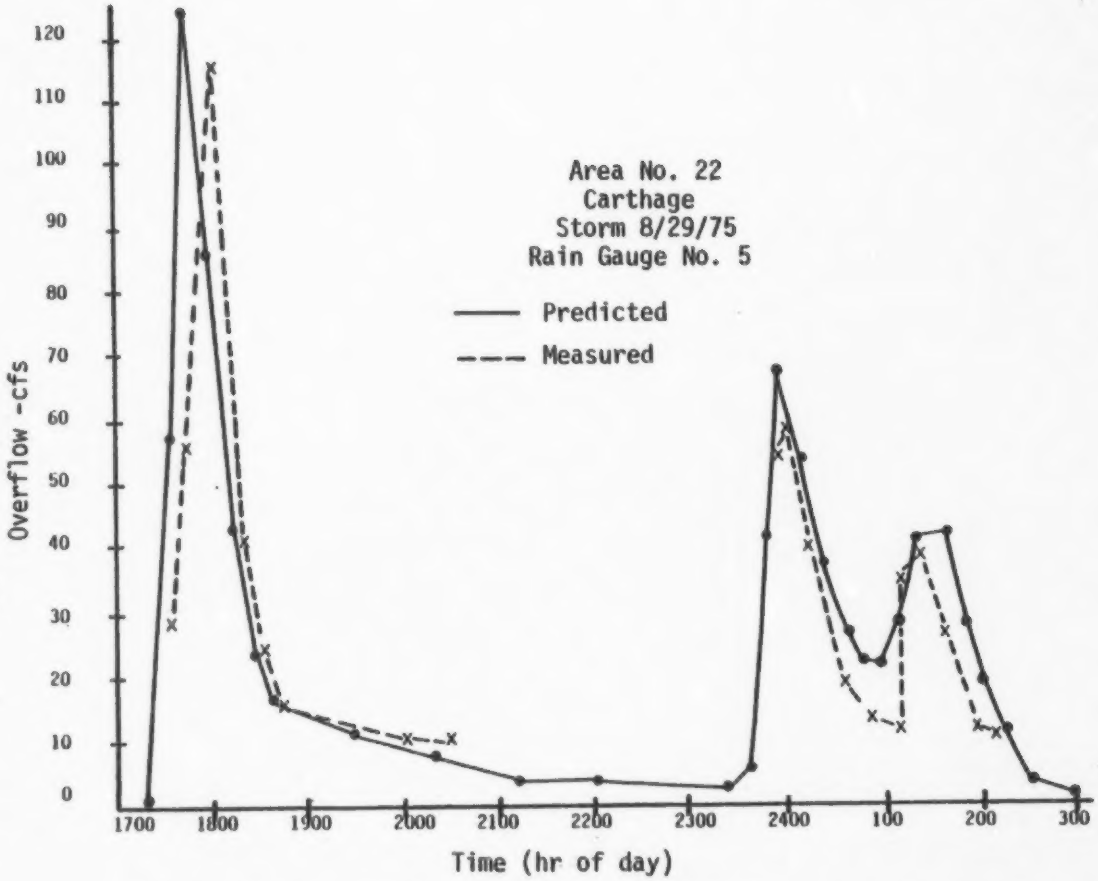


FIGURE 15. Calibration Overflow Hydrograph for Carthage 8/29/75

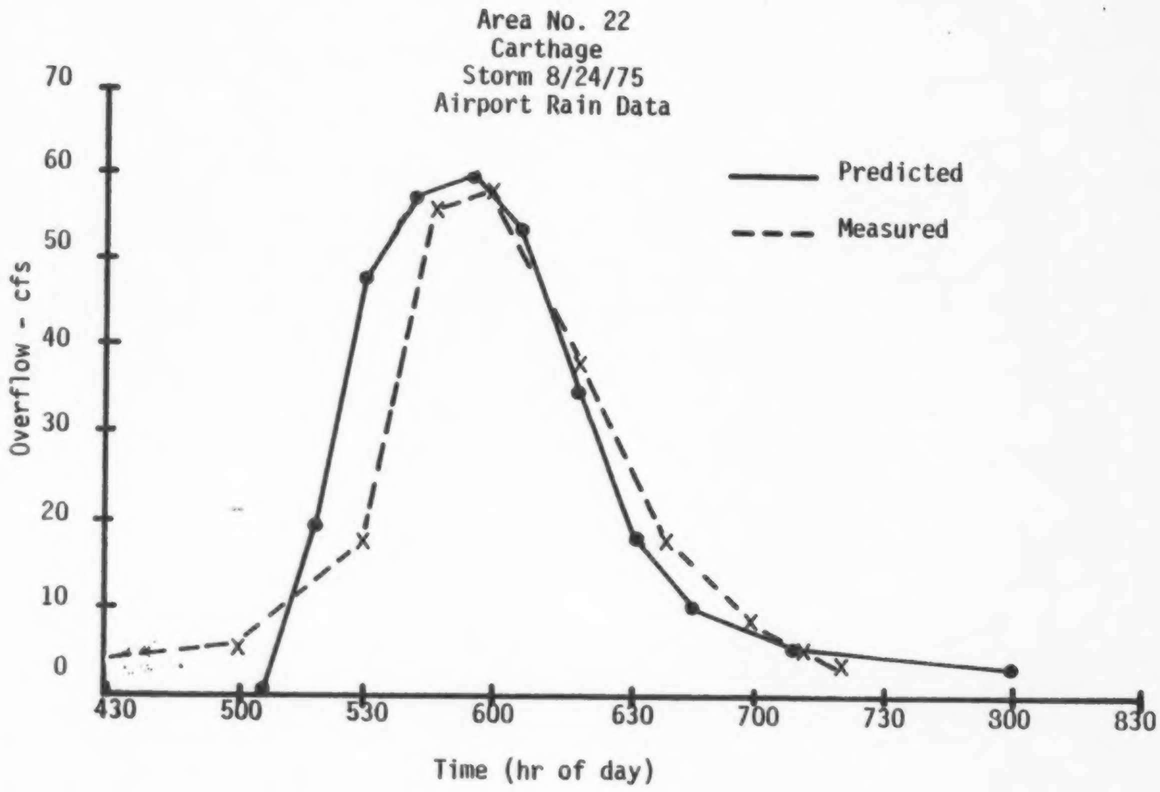


FIGURE 16. Verification Overflow Hydrograph for Carthage 8/24/75

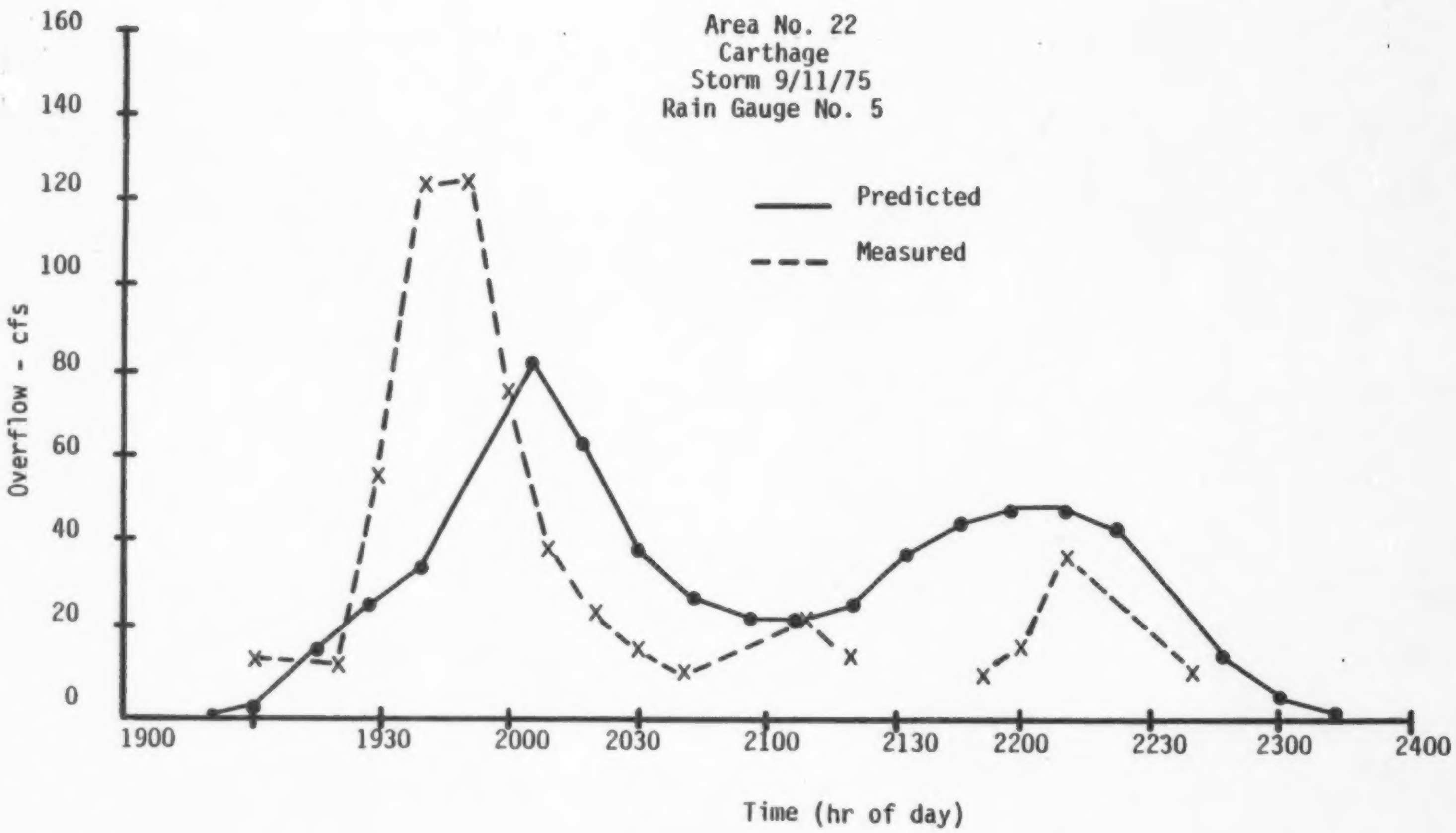


FIGURE 17. Verification Overflow Hydrograph for Carthage 9/11/75

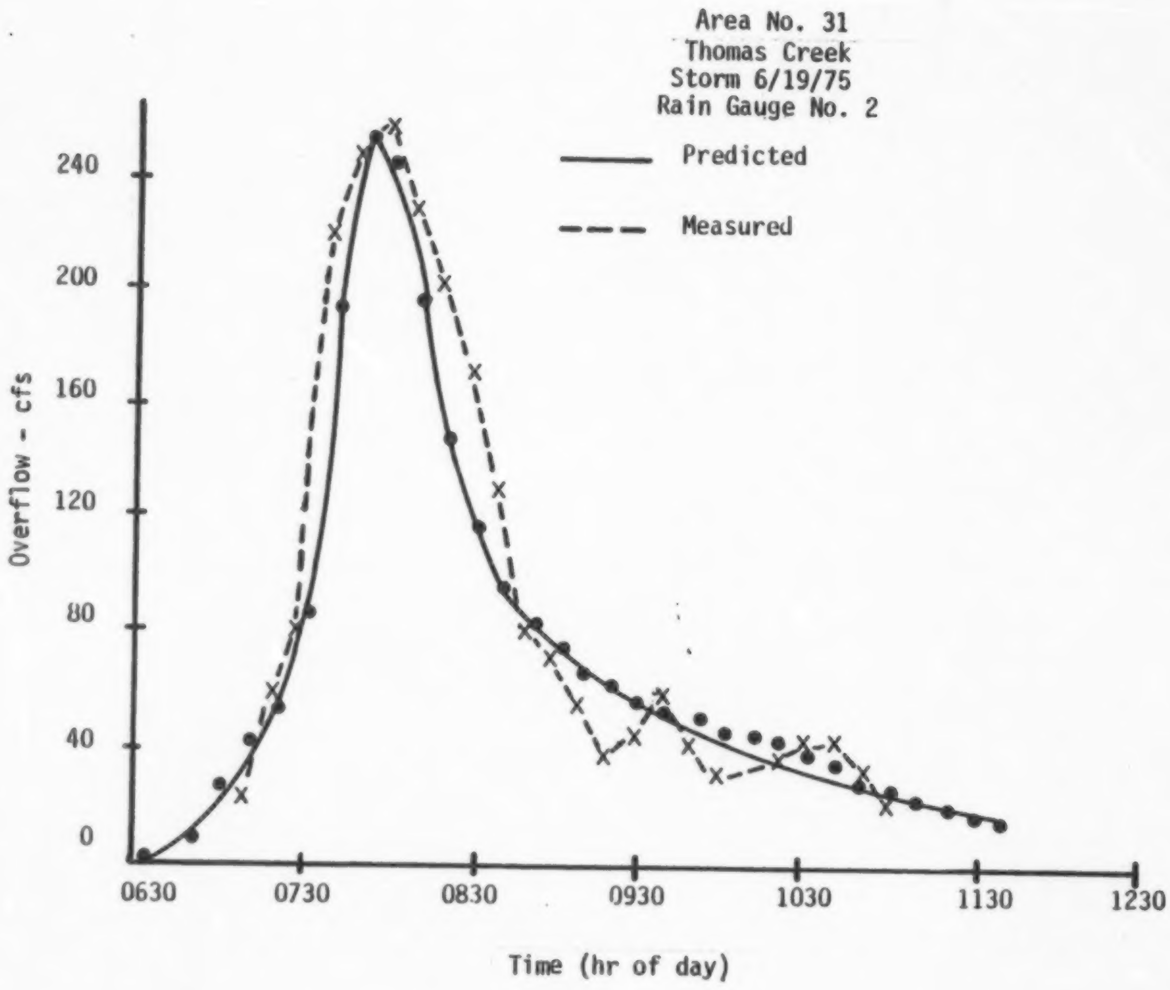


FIGURE 18. Calibration Overflow Hydrograph for Thomas Creek 6/19/75

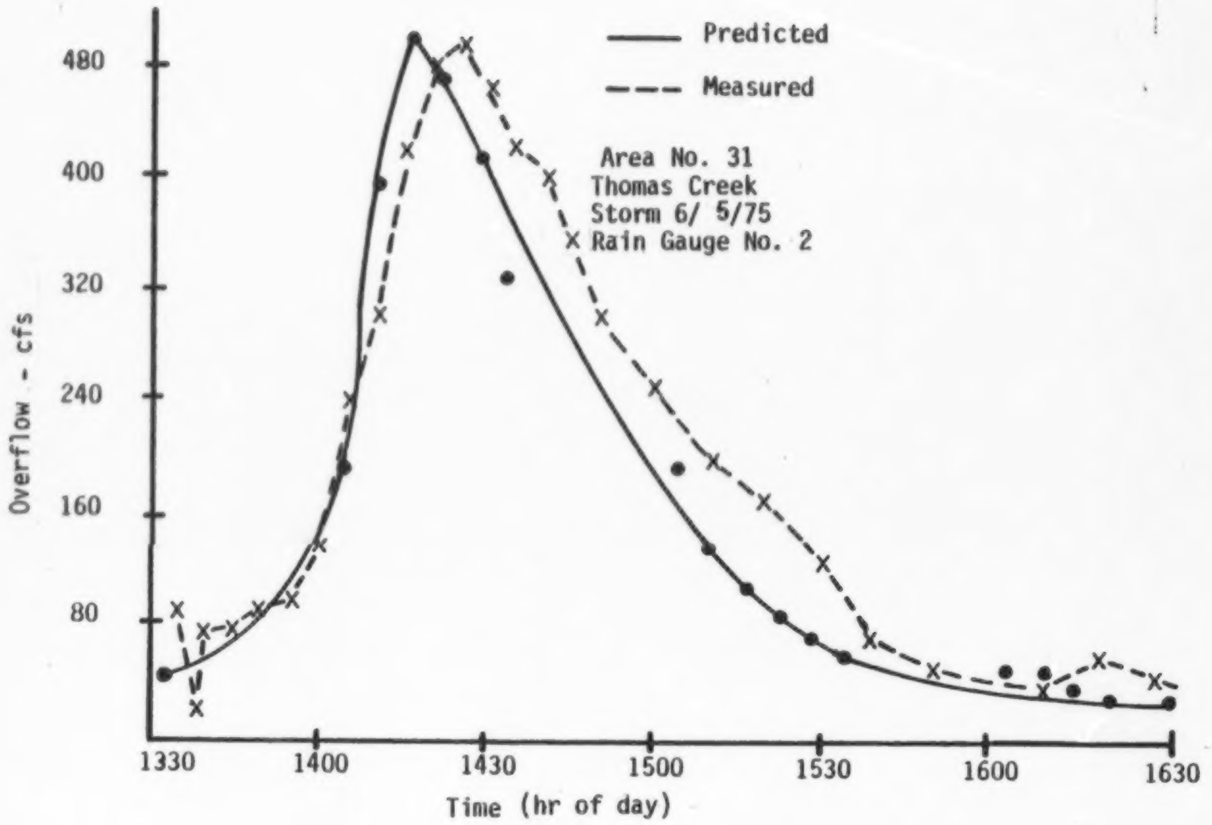
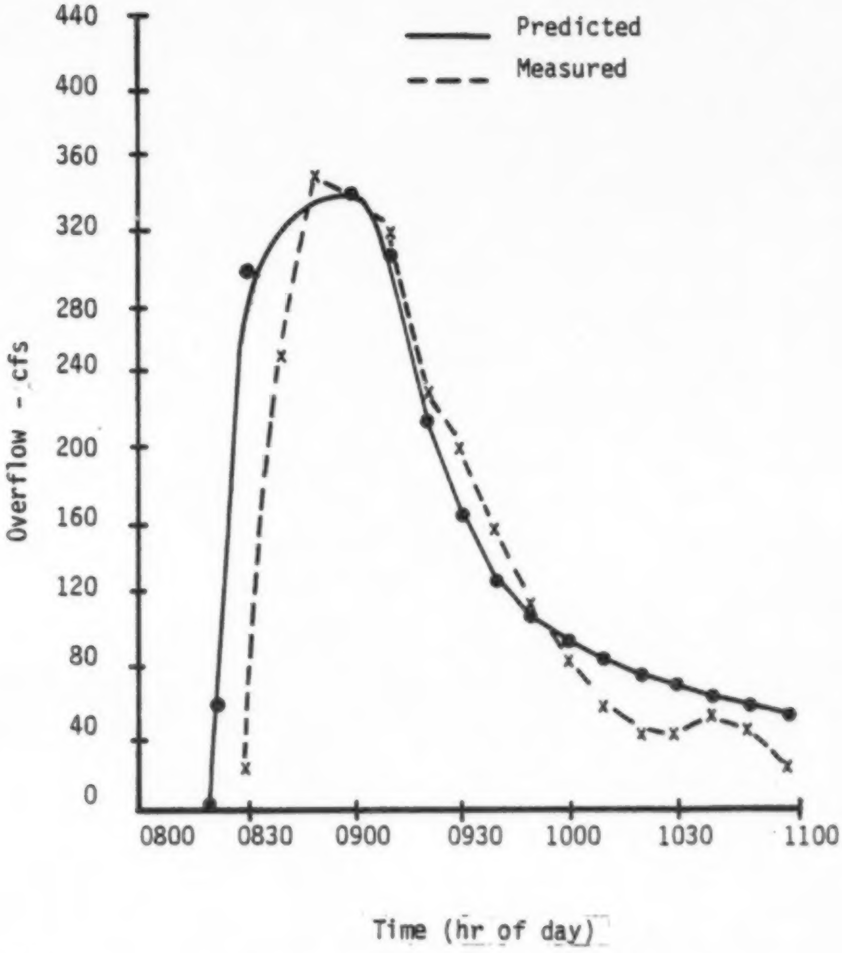
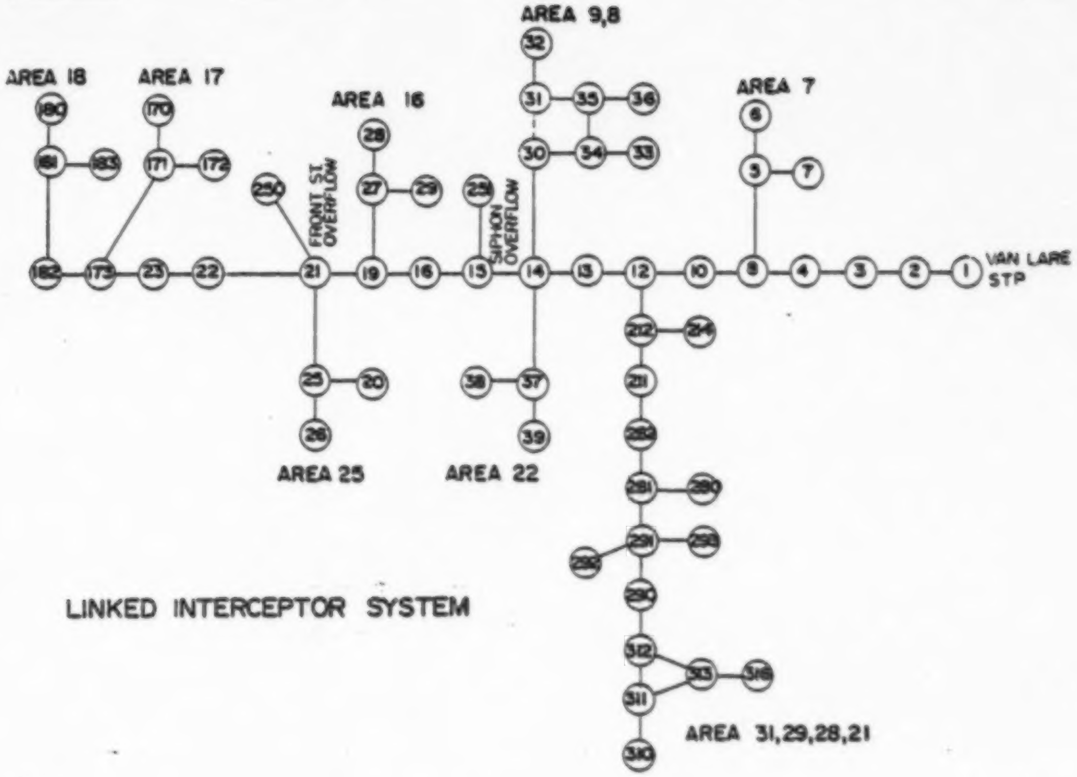


FIGURE 19. Verification Overflow Hydrograph for Thomas Creek 6/5/75



Area No. 31
Thomas Creek
Storm 6/12/75
Rain Gauge No. 2

FIGURE 20. Verification Overflow Hydrograph for Thomas Creek 6/12/75


AREA 9,8
AREA 18
AREA 17
AREA 16
AREA 7
FRONT ST. OVERFLOW
SIPHON OVERFLOW
VAN LARE STP
AREA 25
AREA 22
LINKED INTERCEPTOR SYSTEM
AREA 31,29,28,21
32
31
35
36
30
34
33
180
181
183
170
171
172
28
27
29
250
251
6
5
7
182
173
23
22
21
19
16
15
14
13
12
10
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3
2
1
212
214
211
282
281
280
291
292
293
290
312
313
316
311
310
25
20
26
38
37
39

FIGURE 21. SWMM Node-Conduit Representation of Sewerage Network for City of Rochester

TABLE 6. OVERFLOW PEAK FLOWRATES - VOLUMES

Drainage Area No.	Storm	Peak (cfs)	Site Measured Total Vol. (mil gal)	Peak (cfs)	Model Predicted Total Vol. (mil gal)
7	10/9/75	33	0.86	27	0.85
7	9/11/75	78	2.20	50	1.69
7	9/20/75	65	1.00	72	2.18
22	8/29/75	115	2.30	123	2.51
22	8/24/75	118	3.16	120	3.40
22	9/11/75	124	3.07	82	3.56
31	6/19/75	258	8.17	252	8.02
31	6/ 5/75	490	13.98	495	14.12
31	9/12/75	350	4.36	350	3.98

Note: 1 mil gal = 3785 m^3
1 cfs = 0.00283 m^3/sec

Overflow Quality

The combined sewer overflow pollutants, discharged to the receiving waters during periods of rainfall are generated primarily from surface runoff, although the scouring of accumulated sediments within the sewer conduits may contribute significant amounts of pollutants. In general, the volume of domestic and industrial wastewaters is small in relation to that of surface runoff; however, the associated contribution of bacteria and BOD is large. Projection of overflow quality using the SWMM involves both the runoff and transport routines. In the runoff routine the surface runoff pollutant concentrations are generated based on antecedent rainfall conditions, street cleaning frequency and efficiency, land use, number of catchbasins and length of gutters within the drainage basin. Domestic and industrial wastewater contributions are accounted for in the transport routine of SWMM and are based on land use and population characteristics of the drainage basin. Domestic and industrial wastewater contributions are accounted for in the transport routine of SWMM and are based on land and population characteristics of the drainage basin. The pollutants from both sources are then routed through the sewer system by the transport routine of the model.

Although the overflow quantity verification was quite satisfactory, the quality verification proved to be unsatisfactory. The SWMM runoff routine generates surface pollutants by initially estimating the accumulation of dust and dirt on streets within the drainage area. These accumulated solids also generate an organic demand, which is estimated as a fraction of the dust and dirt present. The pollutant loading rates used in the model were based on studies conducted by the American Public Works Association (APWA) in Chicago (14). From estimates of average daily traffic and average daily litter production, the APWA developed dust and dirt accumulation factors for various types of land use. These loading rates are expressed in terms of pounds of dust and dirt (DD) per day per 100 ft of curb length. The organic demand parameters are in turn generated as a fraction of the DD present. These factors are expressed in terms of milligram of pollutant per gram of DD. More recent versions of SWMM better account for the build-up of surface pollutants.

Since the pollutant loading to a sewer system is largely determined by surface runoff, which is extremely variable with respect to both quantity and quality, modeling efforts to characterize such elements have been difficult at best. The pollutional load from domestic and industrial contributions is relatively easy to identify and quantify. It is, however, the urban surface runoff that largely determines the character of the stormwater or combined sewer overflow discharges.

Poor quality verification has resulted from the use of the SWMM dirt and dust accumulation factors which were not Rochester specific. Although the SWMM program was modified to allow the user to input the values for the DD parameters, it was difficult to establish the correct set of factors for good verification. No extensive studies have been conducted to date in the Rochester area to determine the exact nature of the surface pollutants. The applicability of the SWMM cannot be determined until actual build-up rates have been calculated for these pollutants. Even then, the variability of the contaminants from one location to another could make accurate modeling of runoff quality extremely difficult. Extensive work on quality model calibration provided no reasonable correlation.

In summary, assessment of combined sewer overflow quality can theoretically be addressed by mathematical models such as SWMM. The basic assumption of these models is that the pounds of pollutants washed off in any time interval during a rainfall event is proportional to the pounds remaining on the ground and the runoff rate. In addition, the amount of pollutant accumulation is based on limited studies for a specific area.

A second method, which is more direct and also consistent with the concept of simplified modeling, is the application of regression techniques on data sets collected as part of a combined sewer overflow monitoring program. Under this program the statistical analysis of field observations was used to extrapolate the required quality parameters, after several unsuccessful attempts to modify the quality portion of the SWMM in order to adequately predict the quality characteristics of an overflow. For this study, the linear regression equations were of the form:

$$Q_1 = ax_1 + bx_2 + \ldots + z$$

where Q represents the dependent variable quality parameter, the subscripted X values represent the independent variable quality parameters, and the coefficients a through z represent the calculated regression coefficients. By using a suitable logarithmic transformation of the data, regression analysis can develop equations of the form:

$$Q_1 = x_1^{a} \cdot x_2^{b} \dots z$$

A measure of the ability of the regression equation to predict adequately the dependent variable is the correlation coefficient. The closer the absolute value of the coefficient is to one, the better the correlation between the dependent and independent variables.

From the literature available on the quality of combined sewer overflows, it is known that the quality characteristics of an overflow are extremely variable and are dependent on many factors. The most significant of these factors are assumed to be the intensity of rainfall, the number of antecedent dry days, and street cleaning practices. Land use and population density can also affect overflow quality (15).

Regression equations for the prediction of total suspended solids (TSS) and biochemical oxygen demand (BOD_5) were developed based on several variables including rainfall characteristics and antecedent dry days. The data base consisted of selected measured analytical values from several overflow sites for various storms occurring during 1975. Winter overflow events were excluded because of the possibility of a snowmelt induced overflow independent of precipitation. The data set consisted of the composite TSS and BOD_5 values, the number of dry days (herein defined as the number of elapsed days since the last day having a total of 1.0 in. of rainfall as determined from the rain gauges located throughout the study area), the total duration and average intensity of the rainfall event, the population density, and the percent imperviousness of the drainage area.

Many combinations of variables were selected for regression analysis. Presented in Table 7 are the regression equations having the highest correlation coefficients for TSS and BOD_5. The determined level of correlation

TABLE 7. REGRESSION ANALYSIS - QUALITY PARAMETERS

Total Suspended Solids:

$$TSS = 163.66\ (RAIN)^{-0.424}(DD)^{0.084}(DUR)^{-0.589}$$

Multiple Correlation Coefficient = 0.47

Biochemical Oxygen Demand

$$BOD_5 = 37.83\ (RAIN)^{-0.616}(DD)^{0.159}(DUR)^{-.639}$$

Multiple Correlation Coefficient = 0.61

Where TSS = Total suspended solids - mg/l

RAIN = Average rainfall intensity throughout storm - in./hr

DD = Antecedent dry days in which accumulated rainfall <1.0 in.

DUR = Duration of storm - hrs.

Note: Number of data points = 30

Significance levels on independent variables = 0.95

is limited because of the available data base and the inherent variability of urban stormwater runoff. It is interesting to note that the correlation for BOD_5 is considerably better than that for TSS. Furthermore, the regression coefficients indicate that both TSS and BOD_5 are proportional to the number of antecedent dry days. These relationships were anticipated except for the negative regression coefficient for the rain intensity variable. It would seem that a higher intensity rainfall would impact more energy to surface washoff thereby generating higher concentrations of both TSS and BOD_5.

The samples used in the regression analysis were composited from individual samples taken at uniform intervals from the beginning of the overflow event. A flow-proportional composite sample would correlate quality better with the flowrate and therefore would reflect quality variations related to the flow and would be a more representative approach to quality analysis.

Comparison of SWMM Output To Other Models

A comparison of results from several computer models that simulated the rainfall-runoff relationship for the City of Rochester indicated that relatively simple models can accurately estimate the volume of runoff from a given drainage basin. It was demonstrated that these same models can also reasonably predict the total sewer system overflow volume. Sophisticated models requiring many man-hours of data collection and manipulation and long computer simulation times may not be necessary to closely approximate the volume of runoff and combined sewer overflow discharged from a given drainage area.

The three models used for runoff comparison were the Quantity-Quality Simulation (QQS) by Dorsch Consult (16), SWMM, and SSM. Total runoff volumes generated by the QQS and the one, two, and five year design storm hyetographs, which supplied the rainfall data input, were provided by others (3). The synthetic rainfall hyetographs were formulated using a method developed by Keifer and Chu (17) in conjunction with established rainfall-frequency-duration curves for the Rochester area as obtained from the U.S. Weather Bureau located at the Monroe County Airport. The peak intensity and the total depth of rainfall for each year are shown in Table 8. Furthermore, the same drainage basin characteristics that were used for QQS input were also used as input to the other two models. Runoff volumes generated by the SWMM and SSM were then compared to the QQS results.

TABLE 8. RAINFALL DATA FOR MODEL COMPARISON

Year Storm	Total Depth (in.)	Peak Intensity (in./hr)	Total Duration (min)
1	0.84	2.99	210
2	1.37	3.98	210
5	1.92	5.00	210

Note: 1 in. = 2.54 cm

Although application of the SWMM runoff routine involves estimating seven drainage basin characteristics for data input, only the area in acres, the percent imperviousness, and the maximum and minimum infiltration rates for each drainage area were common to both the QQS and SWMM input data base. Parameters such as subcatchment width, average ground slope, infiltration decay rates, and surface resistance and storage factors were either assumed to be accurately represented by the default values in the SWMM or were estimated from drainage area maps. The entire City of Rochester was modeled using 18 separate drainage areas comprising a total of 12,694 acres. These 18 individual areas were initially further subdivided into smaller subcatchments; however, for the runoff comparison the land characteristics of each subcatchment were area-weighted to provide a single average value for each of the 18 drainage areas. Table 9 lists the basin characteristics for runoff data input for the SWMM for each of these drainage areas.

Application of the SSM involved only three parameters: total depth of rainfall, size of drainage area in acres, and gross runoff coefficient. For this comparison, the runoff coefficient was assumed to be equal to the percent imperviousness of the drainage areas. The runoff determination of the SSM is based on a volumetric balance of total rainfall. Although this model is more useful in analyzing many years of rainfall records, it can be run for a particular rainfall event. For a singular storm event the runoff volumes can be calculated easily without the aid of a computer. Table 10 lists by recurrence interval the runoff volumes as generated by each of the three models.

TABLE 10. RUNOFF VOLUMES FOR MODEL COMPARISONS

	Total Runoff Volume (10^6 ft^3)		
Year Storm	QQS	SWMM	SSM
1	17	16	17
2	29	29	28
5	47	46	39

As indicated, the runoff volumes generated by the QQS and SWMM are essentially the same. Volumes generated by the SSM are similar to those from the other models except for those associated with less frequent storm events; that is, events less frequent than the two-year storm.

On verification of the QQS and SWMM projected runoff volumes, both models were compared on the basis of total system overflow volume to the Genesee River and Irondequoit Bay. System overflow as determined by the SSM was not simulated because of the degree of simplification and assumptions that are required to make the QQS sewer network compatible with SSM input requirements. Because the two-year design storm criteria was to be used in the abatement alternative analysis, only system overflow induced from runoff generated during the two-year storm event was compared.

TABLE 9. SUBCATCHMENT DATA

Subcatch-Ment No.	Gutter Or Inlet	Width (ft)	Area (ac)	Percent Imperv.	Slope (ft /ft)	Resistance Imperv.	Factor Perv.	Surface Storage (in.)		Infiltration Rates (in./hr)		Decay Rate (1/sec)
								Imperv.	Perv.	Maximum	Minimum	
7	5	19600.0	721.	31.5	.030	.013	.250	.062	.184	2.00	.50	.00115
8	210	18000.0	974	42.4								
9	196	45000.0	2172.	40.6								
10	375	14000.0	344.	46.9								
16	303	28000.0	811.	46.7								
15	331	10000.0	511.	25.9								
18	304	5000.0	448.	30.0								
17	316	6000.0	660.	30.0								
19	606	13000.0	57.	85.0								
25	257	15000.0	360.	80.4								
26	3	5000.0	393.	43.3								
310	33	25000.0	1501.	47.8								
311	38	3000.0	170.	68.9								
29	47	23000.0	1353.	54.4								
280	86	10000.0	193.	34.4								
281	90	18500.0	627.	47.2								
81	16	10500.0	838	38.6								
22	128	14000.0	561.	52.5								

Total Number of Subcatchments, 18

Total Tributary Area (Acres), 12694

The primary difference between the sewer network as modeled with the QQS and the SWMM is the degree of detail. The application of the QQS model involved the modeling of hundreds of manholes and sewer conduits throughout each drainage area, whereas, the SWMM, as applied, involved modeling only the main trunk and intercepting sewers. Thus, it has been shown that an accurate determination of total system overflow quantities can be made using a less detailed sewer network.

The sewer network as used in the SWMM simulation is shown in Figure 22. This network was based on data collected in the course of the QQS characterization. Analysis utilizing the SWMM involved 2880 integration steps of 10 seconds each. Table 11 shows the total system overflow volumes as predicted by the two models for the two-year storm.

TABLE 11. OVERFLOW VOLUMES FOR MODEL COMPARISONS

Year Storm	Overflow Volume (10^6 ft^3) QQS	SWMM
2	26	23

Good comparison between the model output for the two-year storm precluded the simulation of the one- and five-year storms. Choosing a smaller time step than 10 seconds, thereby increasing the computer simulation, might possibly increase the accuracy of the SWMM results. Additional runs utilizing a shorter time step seemed unwarranted based on the results for the two year storm.

Computer simulation time for the SWMM runoff block was 0.9 min for each design storm. Modeling of the relatively detailed trunk and intercepting sewer network as shown in Figure 22 required approximately 45 min of CPU computer time on a Xerox 560. Further simplification of the sewer network would allow for shorter computer execution times. If done judiciously, little loss in accuracy would result from such simplifications.

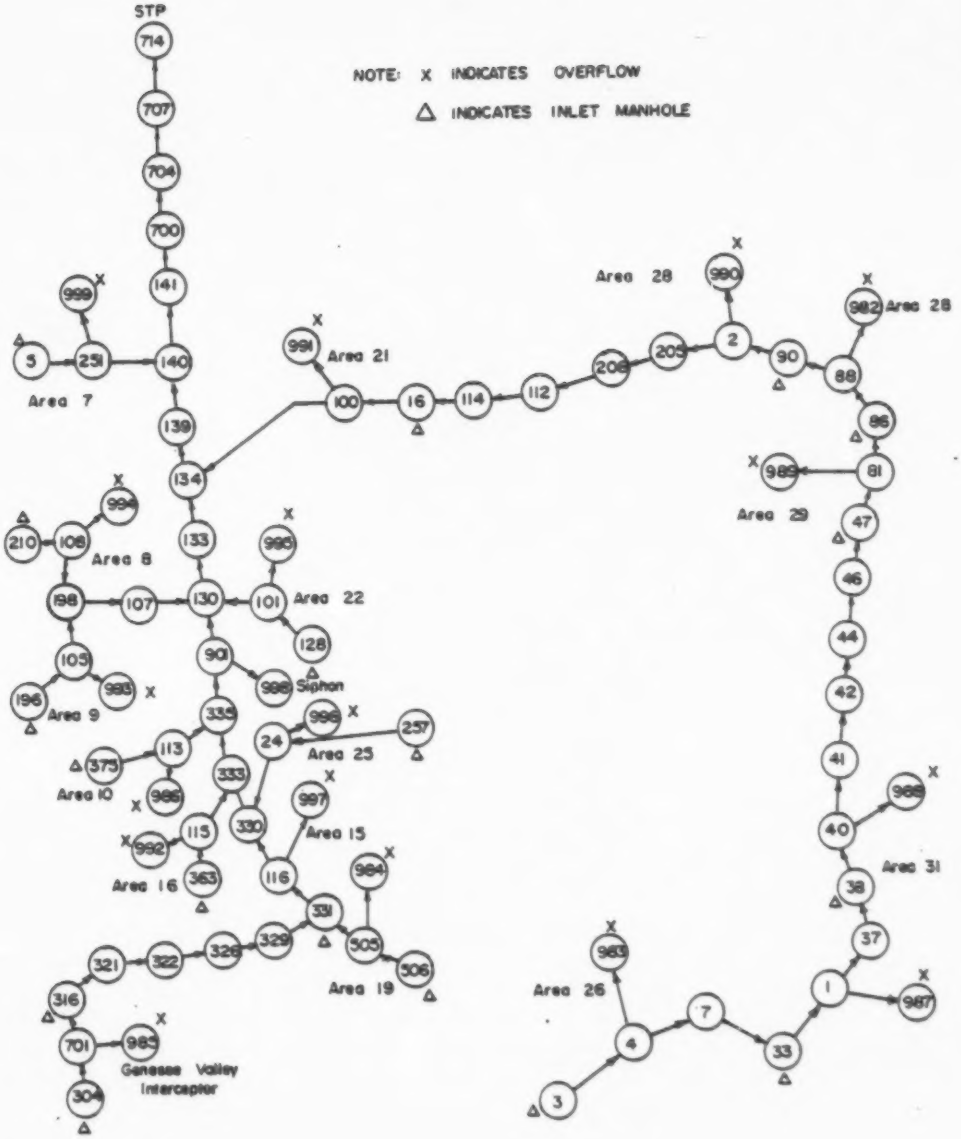

STP
NOTE: X INDICATES OVERFLOW
△ INDICATES INLET MANHOLE
714
707
704
700
141
999
X
5
251
140
Area 7
139
991
X
Area 21
100
16
114
112
208
205
2
990
X
Area 28
90
88
982
X
Area 28
86
81
989
X
Area 29
47
46
44
42
41
40
988
X
Area 31
38
37
1
987
X
33
7
4
983
X
Area 26
3
134
994
X
210
108
Area 8
133
995
X
198
107
130
101
Area 22
128
105
993
X
196
Area 9
901
998
Siphon
335
996
X
24
257
Area 25
113
373
Area 10
986
X
333
997
X
330
Area 15
115
992
X
363
Area 16
116
984
X
331
505
506
Area 19
329
326
322
321
316
701
985
X
Genesee Valley
Interceptor
304

FIGURE 22. SWMM Sewer Network Configuration for QQS-SWMM Comparison

SECTION 7

CRITERIA FOR ALTERNATIVE ANALYSIS

INTRODUCTION

Extensive data on urban storm runoff loadings that were collected over the past several years have been reported in the literature. These provide an immediate source of information and can, when there is a lack of local monitoring and sampling data, provide a first level estimate of the impact of runoff from an urban drainage area. The use of literature derived data is useful in estimating the average yearly stormwater and CSO pollutant loadings. Literature reported data are not, however, a substitute for local data collection and analyses specific to the drainage basin under investigation. Further definition of transient stormwater loads leads to various levels of refinement in pollutant loading projections. The level of refinement generally proceeds as follows:

Level 1 - Average yearly stormwater and combined sewer overflow load

Level 2 - Actual event distribution

Level 3 - Variation within events

The justification for more detailed analyses involving higher levels of sophistication is directly related to the environmental and economic risks involved in decisions made from preliminary analyses involving lower levels of precision and refinement.

The need for adequate local data above that provided by the literature is based on the following factors:

1. Differences in rainfall patterns, frequency, intensity, and duration, in conjunction with surface drainage area characteristics, significantly influence storm runoff volumes and pollutant loadings.
2. Significant variations in storm-generated loads from similar storms occurring at different times throughout the year are observed within the same study area.
3. Studies which attempt to characterize loading data generally cover relatively short periods of time relative to the long term rainfall patterns in a given area.

The most important factor in the generation of storm-induced CSO volumes and pollutant loadings is rainfall. All methods for estimating these storm-induced loadings rely on rainfall data as fundamental input for reasonable and meaningful projections. Much effort has been expended in the field of network modeling and the collection of stormwater and CSO monitoring and sampling. It appears that little effort, however, has been spent in the area of rainfall characterization for abatement analysis. A very difficult, and yet unsolved problem, is the accurate and meaningful development of rainfall hyetographs for various return period frequencies from Weather Bureau frequency-intensity-duration curves or actual local rainfall data.

RAINFALL ANALYSIS

Daily and hourly precipitation records from the U.S. Weather Bureau station at Rochester were used to conduct the network modeling studies. Rainfall records, covering a period from 1948 to 1975, were obtained from the National Climate Center, Asheville, North Carolina. Precipitation patterns and durations for the Rochester area are highly variable. High-intensity, short-duration events are usually associated with thunderstorms occurring during the summer months; whereas, low intensity, long duration events are usually associated with cyclonic activity occurring during the spring and fall months.

Data from local rain gauges located within the study area also provided rainfall information for specific drainage areas which did significantly enhance the developed rainfall hyetographs. A system of twelve rain gauges was installed within the study area of the Rochester Pure Waters District as discussed in Section 5. On the basis of 19 storms recorded between January and August of 1975 the Weather Bureau gauge indicated an average of 0.44 in. per storm while the local gauges recorded an average 0.51 in. The Weather Bureau gauge also indicated an average storm duration of 8.05 hr compared to 5.5 hr, based upon the local gauges. The difference in duration is probably due to the lower sensitivity of the local recording rain gauges. These gauges must accumulate 0.10 in. of rain before indicating the start of a storm event; whereas, the Weather Bureau gauges record to the nearest 0.01 in. Although the local gauges did indicate a rainfall variation pattern throughout the study area for various storm events, they did indicate that rainfall across the District is generally relatively uniform.

Two statistically valid and accepted methods (7, 17) for rainfall characterization were used for abatement alternative analysis. Actual rainfall records for a 20 yr period from January, 1954 to December, 1973 obtained from the U.S. Weather Bureau were used as input to the SSM. Synthetic hyetographs for various return periods derived from Weather Bureau rainfall frequency-duration-intensity curves were used as input to the SWMM.

Based on the 22 years of precipitation records from January, 1954 to December, 1975, Table 12 summarizes the monthly and annual rainfall statistics. In addition, Tables 13 and 14 summarize the number of rain days and the rain per rain day associated with period of record.

TABLE 12. TOTAL RAINFALL (in.)

	Average	Std. Dev.	Coefficient of Variation	95% Confidence Level Interval±
Jan.	2.11	0.87	0.41	0.39
Feb.	2.45	1.10	0.45	0.49
Mar.	2.36	1.01	0.43	0.45
Apr.	2.58	0.83	0.32	0.37
May	2.65	1.20	0.45	0.53
Jun.	2.81	1.61	0.57	0.71
Jul.	2.30	1.23	0.54	0.55
Aug.	3.31	1.26	0.38	0.56
Sept.	2.29	1.14	0.50	0.51
Oct.	2.52	1.81	0.72	0.80
Nov.	2.76	1.12	0.41	0.50
Dec.	2.50	1.10	0.44	0.49
ANNUAL	30.63	4.43	0.14	1.97

TABLE 13. NUMBER OF RAINDAYS

	Average	Std. Dev.	Coefficient of Variation	95% Confidence Level of Interval±
Jan.	15.64	3.16	0.20	1.40
Feb.	14.77	3.70	0.25	1.64
Mar.	13.82	3.62	0.26	1.61
Apr.	12.91	2.74	0.21	1.22
May	11.59	3.42	0.29	1.52
Jun.	9.41	3.22	0.34	1.43
Jul.	9.23	3.12	0.34	1.38
Aug.	10.18	2.13	0.21	0.94
Sep.	10.00	3.80	0.38	1.69
Oct.	11.05	3.12	0.28	1.39
Nov.	15.18	2.92	0.19	1.30
Dec.	17.73	3.15	0.18	1.40
ANNUAL	151.50	12.91	0.09	5.73

TABLE 14. RAIN PER STORM IN INCHES PER RAIN DAY

	Average	Std. Dev.	Coefficient of Variation	95% Confidence Level Interval ±
Jan.	0.14	0.05	0.40	0.20
Feb.	0.17	0.07	0.40	0.03
Mar.	0.17	0.05	0.31	0.02
Apr.	0.21	0.07	0.33	0.03
May	0.23	0.09	0.37	0.04
Jun.	0.29	0.12	0.41	0.05
Jul.	0.26	0.13	0.51	0.06
Aug.	0.33	0.11	0.33	0.05
Sep.	0.23	0.10	0.43	0.04
Oct.	0.21	0.12	0.55	0.05
Nov.	0.18	0.06	0.34	0.03
Dec.	0.14	0.06	0.43	0.03
ANNUAL	0.21	0.02	0.11	0.01

From Tables 12 through 14, it can be seen that monthly precipitation in Rochester is fairly uniform throughout the year. However, as expected, the months of June, July, and August exhibit the higher intensity-shorter duration storm events. During the period of 1943 to 1973, the greatest total amount of rainfall presented by a storm event was 3.91 in. over 87 hr. The greatest maximum hourly intensity was 1.35 in. associated with a duration of 8 hr and a total rainfall depth of 2.15 in.

A rainfall characterization routine in the SSM defines discrete storm events and ranks design parameters associated with each storm. The input data to this program are the hourly rainfall record from the U.S. Weather Bureau. For this study, a discrete storm event has been defined as starting with the first measurable rainfall after a minimum interval of six hours with no rainfall and ending when a gap in measured rainfall of at least six hours is first encountered. For each event in the historical record, the following parameters are calculated: date, starting hour, duration, total rainfall, maximum hourly rainfall and the hour in which it occurred, elapsed hours since the previous storm, occurrences of excessive precipitation, snowfall, and the ratio of hour of maximum rainfall to total duration (r value). Figures 23 through 26 are examples of frequency curves based on ranked rainfall data as provided by the model. Design storm parameters can be determined from these curves. The validity and usefulness of any of these curves is directly related to the length of record available.

In any storm pattern, the three most important characteristics affecting the peak runoff rate for a specific duration are:

(1) volume of water falling within the maximum period,

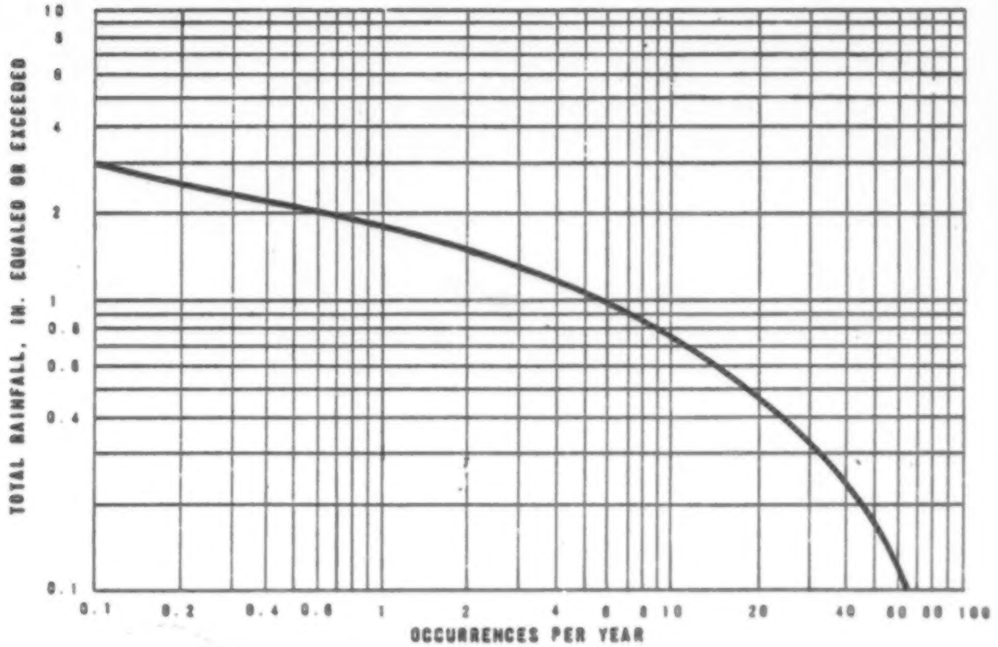


FIGURE 23. Example Curve - Storm Magnitude vs Frequency

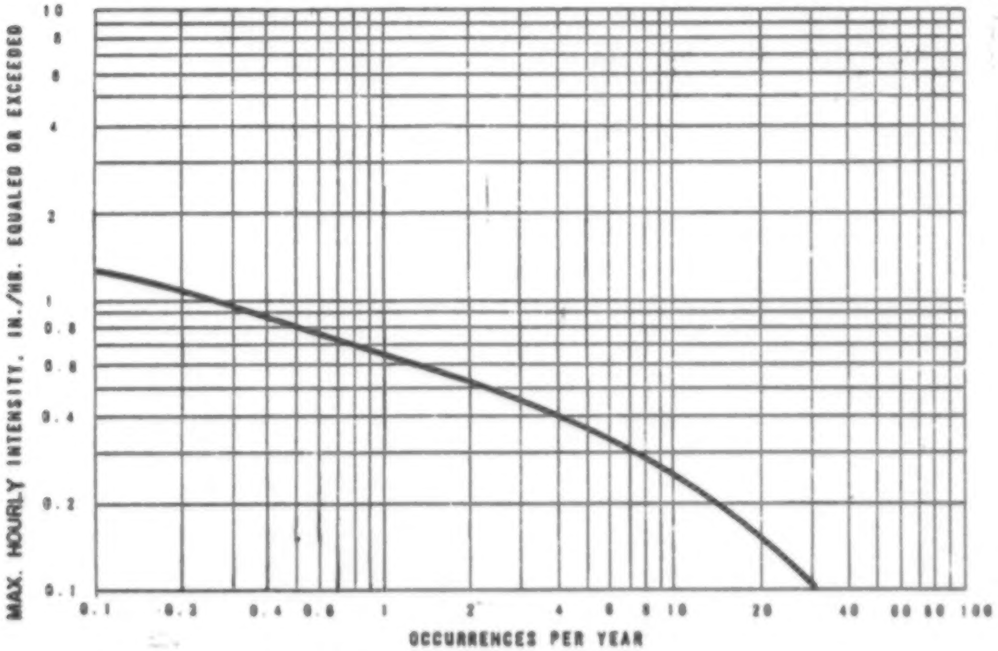


FIGURE 24. Example Curve - Storm Intensity vs Frequency

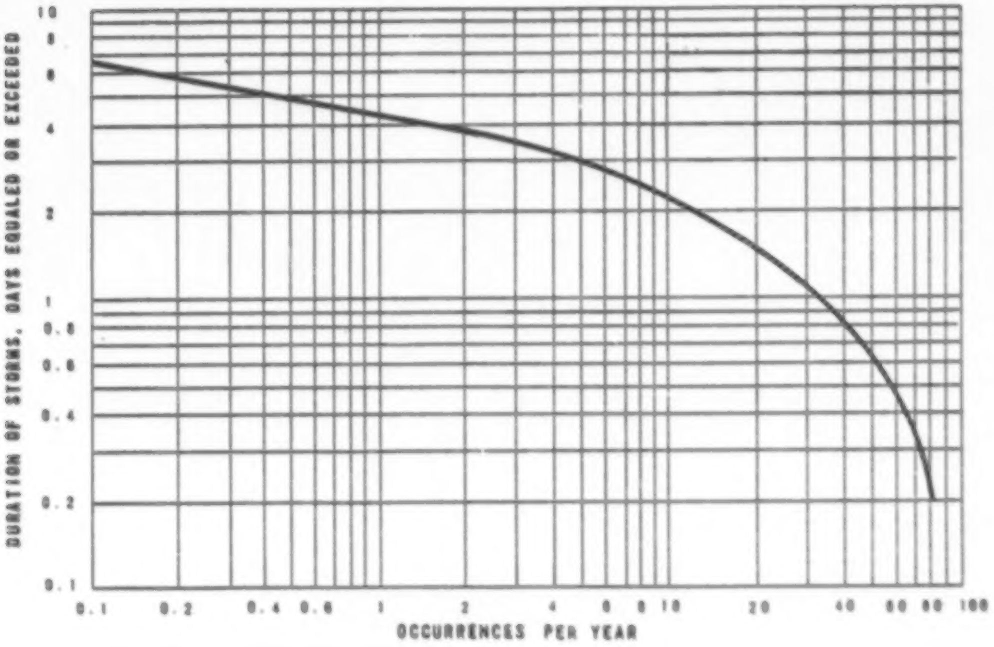


FIGURE 25. Example Curve - Storm Duration vs Frequency

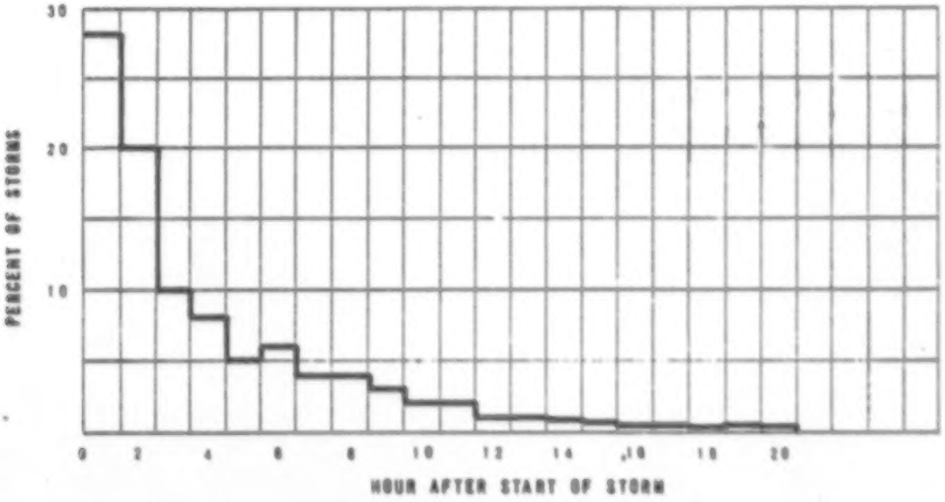


FIGURE 26: Example Curve - Percent of Storms Having Maximum 1-Hour Intensity vs Hour After Start of Storm

(2) amount of antecedent rainfall, and

(3) location of the peak rainfall intensity.

The application of the Keifer and Chu method (7) to produce a given design storm hyetograph requires the consideration of the statistical average of these three characteristics.

For subsequent abatement alternative analyses involving the SWMM, initial runs would involve the one, two, and five year return period design storms utilizing the synthetic hyetographs. A two-hour duration storm event was selected in order to simulate a high-intensity storm. It must be noted that the selection of this specific storm duration was consistent with the selection of the design storm frequency utilized in the ongoing facilities planning activity. Rainfall events of different durations would be expected to differ significantly in their effect on the sewer network. The location of the peak rainfall was determined from a statistical analysis of approximately 180 storms that occurred in the Rochester area between 1933 and 1973 (5).

The equation relating rainfall intensity to duration for any return period can be represented mathematically by: $i = a\,(t^b + c)^{-1}$ where

i = rainfall intensity (in./hr)

t = duration time (min)

a, b, c = constants, dependent on specific region and terrain.

Table 15 summarizes the equation constants for Rochester, New York for various return periods.

TABLE 15. RAINFALL INTENSITY-DURATION EQUATION CONSTANTS

Return Period in Years	a	b	c
1	28	0.85	7.0
2	42	0.88	7.7
5	68	0.90	10.0

Figure 27 shows the rainfall intensity-frequency-duration curves for Rochester, New York.

The general equation used for generating synthetic hyetographs is as follows:

$$i = a\left[(1-b)\left[\frac{t}{r}\right]^b + c\right] \Big/ \left[(t/r)^b + c\right]^2$$

where the variables t, a, b, and c represent the same parameters as in the intensity-duration equation. The term r represents that portion of a storm duration occurring before the most intense moment, expressed as a ratio to the entire duration. A value of r = 0.5 was estimated for the Rochester study

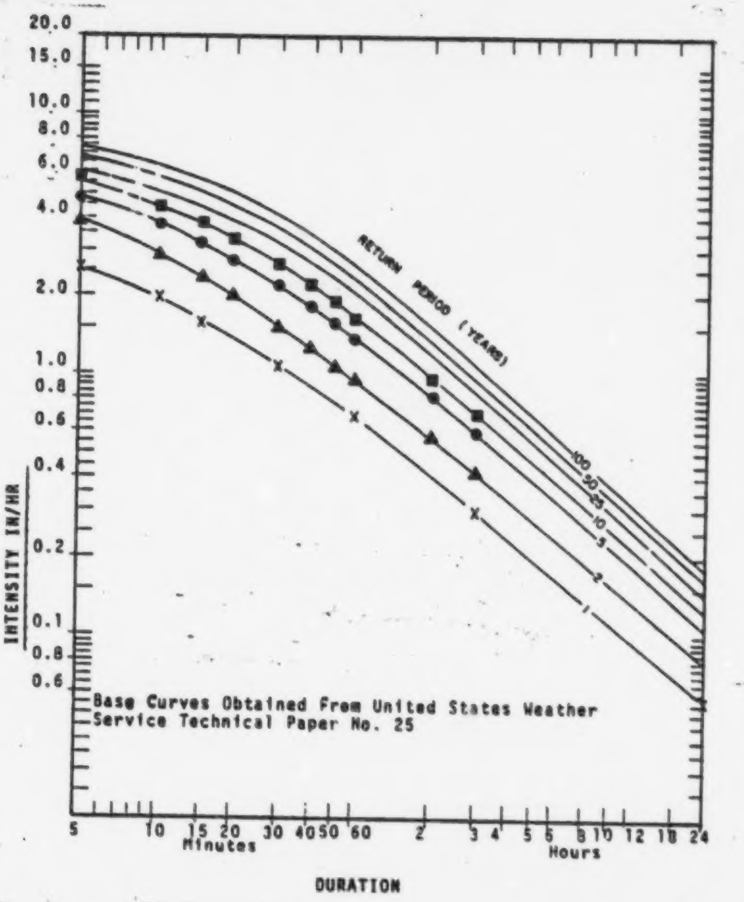


FIGURE 27. Rainfall Intensity-Duration-Frequency Curves For Rochester, N.Y.

area based on the statistical analysis conducted on the recorded 180 storms. Figure 28 shows the generated synthetic hyetographs for the one, two, and five year design storms.

SWMM Version II is a specific storm event model with its simulation dependent on single input rainfall hyetographs. To assess the various CSO abatement alternatives on a return period basis,it was necessary to characterize rainfall events on the same basis. The SWMM overflow projections are based on the one, two, and five year design storms. All average annual overflow volumes predicted through the use of the SSM are based on 20 years of daily rainfall data. The use of hourly rainfall data provides greater accuracy in predicting overflow quantities; however, the SSM is not ideally structured to analyze an entire sewer system using hourly data. Several runs were made to compare the results obtained from daily and hourly rainfall data. A reasonable first-cut analysis of a CSO system can be made with the SSM using daily data.

For all subsequent analyses involving overflow quality and abatement alternative evaluations a 2 Yr-2 Hr design storm was selected. The use of the 2 Yr rainfall return frequency was consistent with other ongoing abatement programs for Monroe County, New York (3). The 2 Hr duration was selected because the times of concentration for all tributary drainage areas to the main interceptor are less than 2 hr and thus all of the associated area would be contributing runoff at its maximum rate which was important for SWMM runoff projections. In addition, it was felt that the subsequent abatement analyses would be most effective when alternatives are based on average storm durations; that is, alternatives that reduce CSO pollution for most storm events.

OVERFLOW QUALITY ANALYSIS

A summary of all the CSO monitored quality data for BOD_5, SS, and t. coliform collected at thirteen overflow sites during 1975 is presented in Table 16. This table also presents the basic CSO quality statistical data which are basic to the following discussion.

The thirteen sites represent those monitored overflow locations shown in Figure 8. The relationship of the overflow sites to each of the major trunk sewers and interceptors of the sewer system for the Rochester Pure Waters District is also shown.

Although the magnitudes of the arithmetic and geometric means vary considerably, relative to the entire drainage area of the Rochester Pure Waters District, quality parameters of BOD_5, SS, and t. coliform are reasonably well defined by either of the two means. As an example, the ratio of the arithmetic mean of BOD_5 for drainage area no. 22 with respect to the average BOD_5 of all drainage areas is 4.21; whereas, the same ratio using geometric means is 4.17. It can be reasonably inferred that the average BOD_5 concentration of drainage area no. 22 is approximately 4.2 times greater than the overall average drainage basin BOD_5 mean. The other drainage areas exhibit similar consistency between their arithmetic and geometric means. Expressed mathematically,

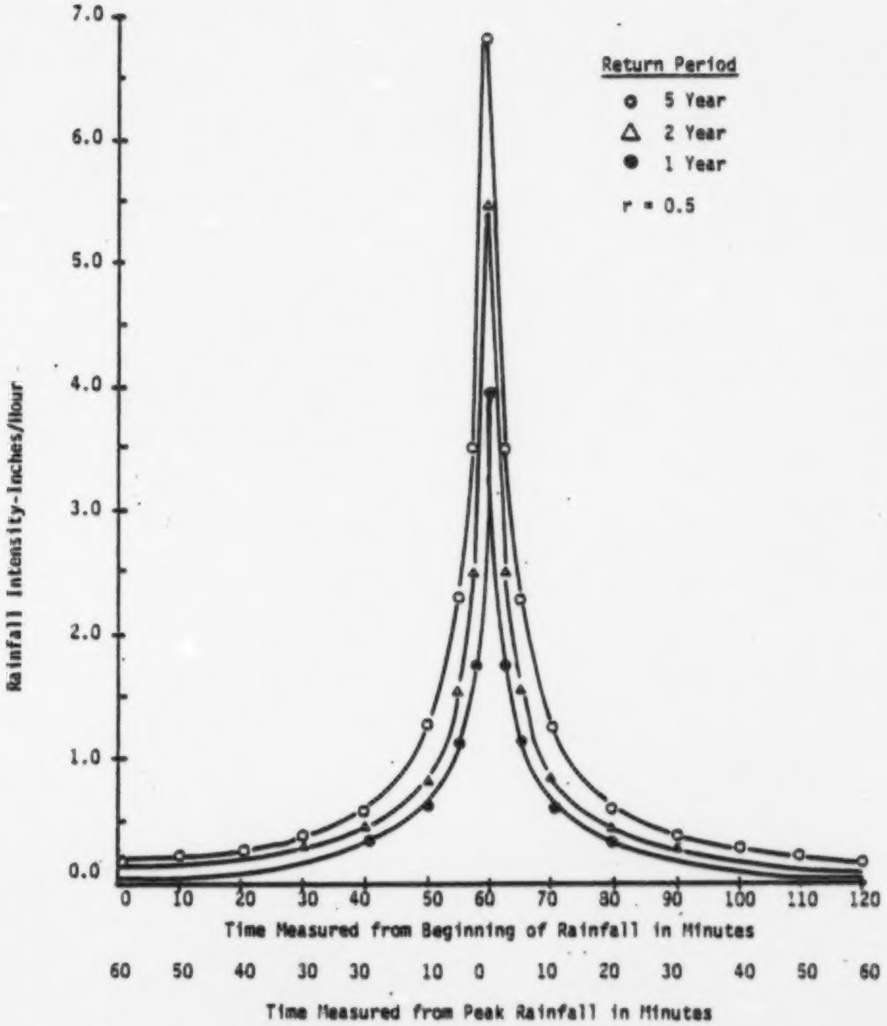


FIGURE 28. Synthetic Design Storm Hyetographs

TABLE 16. SUMMARY OF DRAINAGE AREA CSO QUALITY

Drainage Area	BOD_5 (mg/l) Arithmetic Mean	BOD_5 (mg/l) Geometric Mean	Z	SS (mg/l) Arithmetic Mean	SS (mg/l) Geometric Mean	Z	(MPN/100 ml)x10^{-6} Arithmetic Mean	(MPN/100 ml)x10^{-6} Geometric Mean	Z
7	134 (±28)**	79	1.18	449 (± 80)	247	1.28	5.1 (±1.4)	1.7	0.84
16	97 (±31)	74	0.86	474 (±264)	306	1.35	7.2 (±2.9)	4.5	1.18
22	478 (±49)	308	4.21	591 (± 65)	438	1.68	8.3 (±1.3)	3.9	1.36
8	69 (±18)	47	0.61	169 (± 15)	111	0.37	2.0 (±0.5)	1.0	0.33
9	130 (±29)	118	1.15	220 (± 43)	200	0.63	10.2 (±6.6)	4.4	1.67
17									
18	63 (±11)	44	0.56	194 (± 48)	106	0.55	7.1 (±1.4)	4.0	1.16
21	136 (±86)	75	1.20	511 (±338)	269	1.46	14.1 (±7.7)	9.0	2.31
25	61 (±21)	49	0.54	258 (± 67)	212	0.74	2.0 (±1.5)	0.8	0.33
26	15 (± 4)	11	0.13	644 (±790)***	121	1.83	0.3 (±0.1)	0.2	0.05
28	78 (± 9)	57	0.69	331 (± 52)	193	0.94	5.9 (±1.2)	2.4	0.97
29	31 (± 9)	25	0.27	235 (± 57)	197	0.67			
31	69 (±10)	61	0.61	177 (± 17)	137	0.50	5.2 (±0.8)	3.1	0.85
Y	113			351			6.1		

Y = Average of Arithmetic Means
Z = Mean Value/Y
* = Refer to Figure 8 for location of drainage areas
** = Values in parenthesis represent 95% confidence intervals
*** = Value not used in analysis of arithmentic and geometric mean comparison

$$Z_{1 \cdot N} = X_{1 \cdot N} / Y_1 \quad \text{and}$$

$$Z_{2 \cdot N} = X_{2 \cdot N} / Y_2$$

where

X = arithmetic mean of a specific quality parameter

Y = average of all drainage area arithmetic means (for a particular quality parameter)

Z = characteristic ratio of drainage area mean to overall drainage basin mean

N = number of the drainage area

1 = arithmetic mean

2 = geometric mean

The 95% confidence intervals for the ratio Z_1/Z_2 are shown in Table 17.

TABLE 17. Z_1/Z_2 CONFIDENCE INTERVALS

Quality Parameter	95% Z_1/Z_2 Confidence Intervals
BOD_5	0.96 ± 0.06
TSS	0.88 ± 0.10
t. coliform	1.07 ± 0.14

Several conclusions were derived from analyzing the overflow monitoring data:

1. Drainage areas nos. 7, 21, and 22 have annual mean BOD_5 and SS concentrations exceeding the mean for the overall drainage basin.

2. Drainage areas nos. 21 and 22 have annual mean concentrations for all three quality parameters exceeding the basin average.

3. Pollutant concentrations from overflows discharging to Irondequoit Bay are significantly lower than most of those overflows discharging to the Genesee River.

4. River overflows nos. 8 and 25 similarly exhibit lower concentrations than those of the remaining River overflows.

5. Most overflows except nos. 8, 9, and 22 exhibit SS concentrations on the order of twice the BOD_5 concentrations.

6. In general, those drainage areas with population densities of approximately 18 people per acre and greater, show a t. coliform concentration mean greater than the overall basin average.

The data presented in Table 16 in conjunction with the overflow volumes predicted by the SWMM under the application of the synthetic 2 Yr-2 Hr design storm lend insight to the pollutional impact that the various overflows have on the Genesee River and Irondequoit Bay. Table 18 lists the volume and pollutional load placed on the receiving waters by each overflow.

The total mass of pollutants discharged to the Genesee River was computed from the volume of overflow over 30 min periods, as determined by the model projected hydrographs, and the concentration of these pollutants averaged over the same time periods, as determined from actual monitored data. This procedure is described in the following section. The overflow identified as the Siphon represents a relief point on the main interceptor. The ranking of the individual drainage areas with respect to pollutant loads per volume of overflow and per acre associated with each individual drainage area is presented in Table 19. These rankings indicate that the predominately residential areas contribute a lower areal mass loading of pollutants than industrial and commercial areas. The relatively large pollutional loads imposed by drainage area no. 22 result from the location of a large brewery in this area discharging directly into the District sewerage system without pretreatment.

TABLE 19. RANKING OF DRAINAGE AREAS PER MASS LOADINGS

Drainage Area No.	Predominant Land Use	Ranking - % of River Overflow	Ranking - BOD_5 Loading lb/ac	Ranking - TSS Loading lb/ac
8,9	Residential	1	4	5
7	Residential	2	5	4
25	Commercial	3	3	2
16	Commercial	4	2	3
21	Residential	5	6	6
22	Residential	6	1	1

QUALITY ANALYSIS FOR USE IN ABATEMENT ALTERNATIVE STUDIES

Before an abatement alternative can be evaluated, the pollutional load imposed on the receiving waters must be determined. Analytical data from the sampling of the overflows generated in 1975 were statistically analyzed to yield geometric mean concentrations of selected pollutant parameters within pre-determined time intervals after the start of an overflow event. The time interval selected for analysis was 30 min which yielded three intervals for the first 90 min of the overflow and a final interval extending from 90 min to the end of the entire overflow event.

A 30 min interval was selected on the basis of the 15 min sampling frequency conducted under the monitoring program in conjunction with the

TABLE 18. DRAINAGE AREA OVERFLOW ANALYSIS FOR 2YR-2HR DESIGN STORM

Drainage Area No.	Overflow (mil gal)	% of Total Overflow	% of River Overflow (101.2 mil gal)	Total BOD_5 (lb)	BOD_5 (lb/acre)	Total SS (lb)	SS (lb/acre)
7	9.1	6.4	9.0	6,000	6.7	18,750	20.4
16	8.5	6.0	8.4	5,250	10.1	21,690	28.7
22	5.8	4.1	5.7	14,900	24.0	21,190	41.9
8 & 9	41.4	29.0	40.9	25,710	8.1	51,450	16.3
Front St.*	6.5	4.6	6.4	2,390		5,750	
Siphon*	13.0	9.1	12.8	4,770		11,490	
21	8.4	5.9	8.3	5,250	3.8	18,850	6.4
25	8.5	6.0	8.4	3,470	8.5	15,030	36.8
28	10.1	7.1					
29	20.9	14.6					
31	10.5	7.4					
Totals	142.7			67,740		164,200	

*Drainage areas identified as nos. 17 and 18 in Table 16 have been deleted since overflows from these two basins have been eliminated due to a new interceptor. Front St. and Siphon represent overflows/locations that were not field monitored under this program. BOD_5 and SS loadings based on concentrations (geometric means) presented in Table 16 and volumes based on projections by the SWMM.

assumption that large concentration changes can be best represented by such an interval. This statistical analysis provided average concentration profiles for BOD_5 and TSS to be expected for each overflow event at each point of discharge. Due to the large amount of analytical data collected during 1975, it is assumed that for all the storm event evaluations considered in this study, the calculated concentrations for the selected time intervals remain reasonably constant. Table 20 shows the pollutant concentrations as determined for each drainage area.

TABLE 20. OVERFLOW QUALITY BY TIME INCREMENT

Drainage Area No.	Concentration (mg/1)							
	0-30 min		30-60 min		60-90 min		90+ min	
	BOD_5*	TSS*	BOD_5*	TSS*	BOD_5*	TSS*	BOD_5*	TSS*
7	141	316	83	294	43	123	44	95
8	135	211	58	133	49	129	43	144
9	146	200	176	228	147	198	109	200
16	124	228	173	822	105	508	38	241
17	131	114	-	-	129	357	22	60
18	112	211	104	291	54	154	35	88
21	67	119	38	83	35	62	50	70
22	342	586	268	476	213	392	210	332
25	47	238	33	189	47	189	39	84
26	24	238	20	353	*	198	*	75
28	51	194	41	181	28	178	40	154
29	19	121	27	104	27	120	*	81
31	152	191	47	168	71	174	*	100

BOD_5 and TSS are geometric means in mg/1 from the 1975 analytical data.
*Indicates zero values(s) in data set, thus geometric mean not valid.

From Table 20, the conditions known as first-flush, in which a disproportionately high pollutional load is carried in the first portion of an overflow, is generally observed in each of the drainage areas. Based on an analysis of the data, the first-flush portion of the overflow event has been defined as the first 60 min. From the concentrations and volumes of overflow the total pollutant load to the receiving waters from each overflow site can be determined. Table 21 shows the overflow volumes within the 30 min time intervals determined by the SWMM under the application of the 2 Yr-2 Hr design storm.

The sewer system network modeled included the existing sewer system with the new Genesee Valley Interceptor (and subsequent overflow from drainage areas nos. 17 and 18), the Genesee River Southeast Interceptor Tunnel, and the Culver-Goodman and Cross-Irondequoit Tunnel complex as identified in Figure 29. Since sampling and monitoring were not conducted at the Front Street and Siphon overflow sites, overflow volumes by time interval were unnecessary. In addition, since all the overflow discharging from drainage areas nos. 28, 29, and 31 will be conveyed to treatment facilities by the Cross-Irondequoit Tunnel, it was considered

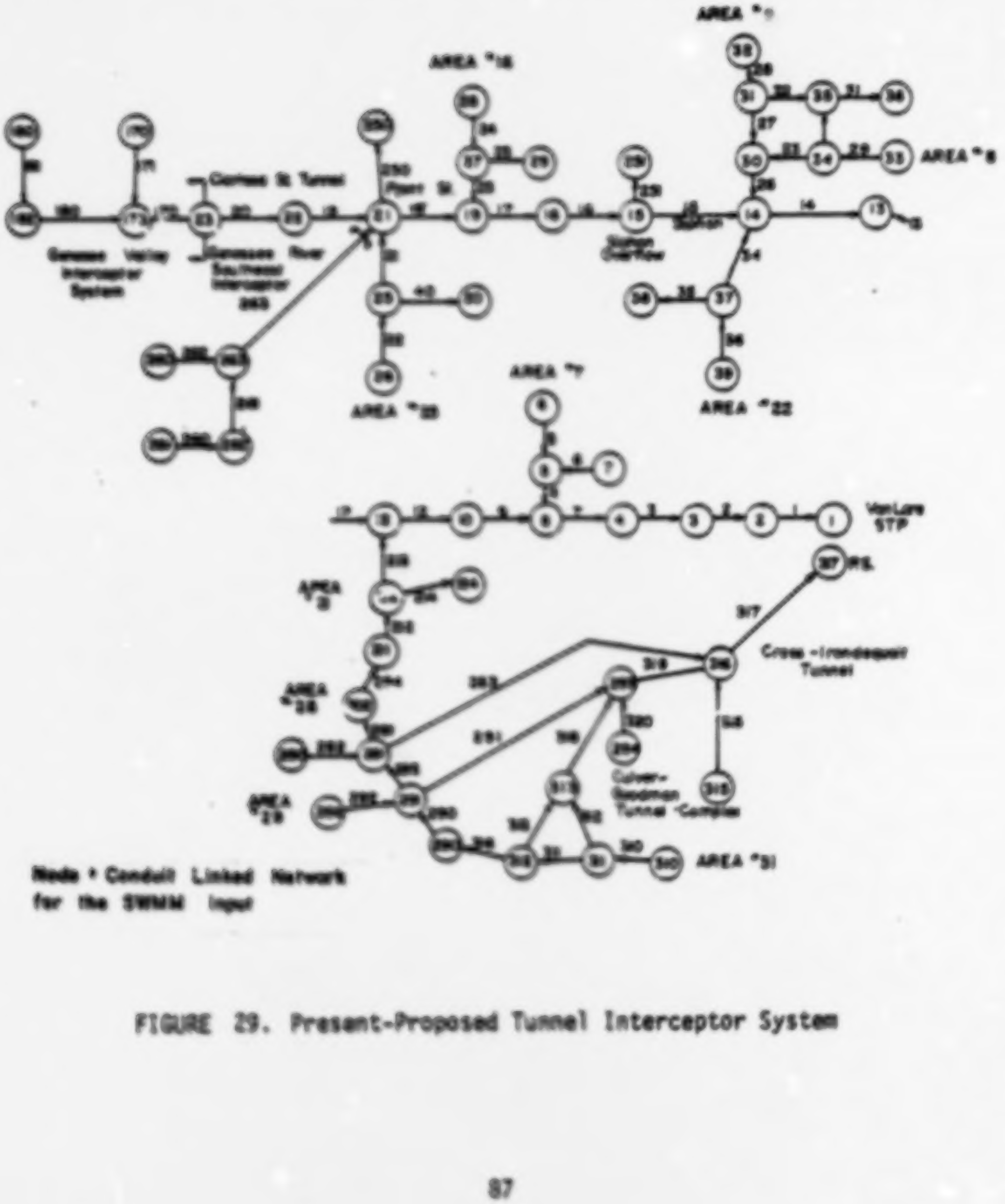

AREA #18
AREA #9
AREA #8
Genesee Valley Interceptor System
Genesee River Southeast Interceptor
Platt St.
AREA #23
AREA #22
AREA #7
Van Lare STP
P.S.
Cross-Irondequoit Tunnel
Culver-Goodman Tunnel Complex
AREA #28
AREA #29
AREA #31
Node + Conduit Linked Network for the SWMM Input

FIGURE 29. Present-Proposed Tunnel Interceptor System

sufficiently accurate to use mean concentration values for the entire storm event and total overflow volumes to determine their pollutant loading for further abatement alternative analyses.

TABLE 21. OVERFLOW VOLUME BY TIME INCREMENT
2 YR - 2 HR DESIGN STORM

Drainage Area No.	Overflow Volume (mil gal) 0-30 min	30-60 min	60-90 min	90+min
7	0.48	3.43	3.26	1.92
8,9	2.88	10.01	9.85	18.67
16	2.38	3.62	2.00	0.51
21	0.54	1.89	2.06	3.94
22	2.21	2.36	0.86	0.37
25	1.80	3.28	1.54	2.28
28	1.21	4.64	2.29	1.91
29	6.88	10.71	2.65	0.64
31	2.09	5.89	1.80	0.72

Table 22 shows the BOD_5 and TSS loadings, by time intervals, discharged to the River from overflows induced by application of the design storm. Since drainage areas nos. 8 and 9 have a common overflow discharge conduit, the loadings from each area were combined by using the geometric mean of the individual area means for the pollutant concentrations. Total loadings were given for Front Street and the Siphon, based upon concentrations of 30 mg/l BOD_5 and 89 mg/l TSS, and 65 mg/l BOD_5 and 273 mg/l TSS, for each overflow site, respectively. The concentrations for Front Street represent approximate means of the concentrations from overflows from drainage areas nos. 17, 18, and 26. With the elimination of the overflows from areas nos. 17 and 18, upon completion of the Genesee Valley Southwest Interceptor, the first relief point on the system will then be at Front Street. Concentrations for the Siphon overflow were assumed similar to those from overflow no. 16.

TABLE 22. OVERFLOW POLLUTANT LOADINGS BY DRAINAGE AREA AND TIME INCREMENT
2YR-2HR DESIGN STORM

	Loadings (lb) 0-30 min		30-60 min		60-90 min		90+ min	
Drainage Area No.	BOD_5	TSS	BOD_5	TSS	BOD_5	TSS	BOD_5	TSS
7	560	1270	2370	8410	1170	3340	700	1520
8,9	3360	4920	8430	14530	6900	13140	10590	26310
16	2460	4530	4020	9720	1730	8470	160	1030
21	300	540	600	1310	600	1070	1640	2300
22	6300	10800	5270	9370	1530	2810	650	1020
25	710	3570	900	5170	600	2430	740	1600
Front Street	Loadings in text							
Siphon								

Drainage Area No.		Total Loadings (lb)	
		BOD$_5$	TSS
7		4800	14540
8,9		29280	58900
16		8370	23750
21		3140	5220
22		13750	24000
25		2950	12770
Front St.		1620	4790
Siphon		7020	29480
	Totals	70930	173450

Presented in Table 23 are the loadings to the Cross-Irondequoit Tunnel from the East Side Trunk Sewer overflows for the defined design storm.

These loadings are based on data from Tables 20 through 22. The total combined sewer overflow loading to the Genesee River for the 2 Yr-2 Hr design storm is 70,930 lb BOD$_5$ and 173,450 lb TSS. Pollutant loading representing the first-flush is 43,920 lb BOD$_5$ and 74,140 lb TSS. The overflow volume associated with the first-flush is 34.88 mil gal.

For a level one analysis associated with subsequent screening of possible abatement alternatives, it was advantageous to determine the average annual BOD$_5$ and TSS loadings to the Genesee River. The Simplified Stormwater Model is ideally suited to calculate such quantities. Twenty years (1954-1973) of daily precipitation records formed the basis for model simulation. The sewer network on which the model was applied was similar to that analyzed using the SWMM in terms of in-system storage volumes and interceptor conveyance capacities. Pollutant concentrations that were associated with overflow volumes establishing mass loadings were based on the 1975 analytical data base. Geometric means of samples taken throughout the duration of the overflow event were assumed sufficiently accurate to be coupled with overflow volumes to yield the mass loadings of the pollutants. Sensitivity analyses conducted on the SSM using both daily and hourly rainfall data indicated that the total overflow volume discharged to the Genesee River as a result of daily rainfall data should be increased by 60 percent to more accurately represent actual system rainfall-runoff-overflow response as determined by application of hourly data. Table 24 shows the results of the simulations and subsequent projections made with the SSM.

Assuming a total urban tributary drainage area to the Genesee River of 9172 acres and assuming a gross runoff coefficient of 0.431, the average annual rainfall for Rochester of 30.67 translates into approximately 3300 mil gal of runoff. Therefore, based upon the projections using the SSM, one-third of all rainfall becomes combined sewer overflow with subsequent discharge to the River while only 10 percent of the total rainfall remains in the sewer system for possible treatment under present conditions.

TABLE 23. POLLUTANT LOADINGS BY TIME INCREMENT 2YR - 2HR DESIGN STORM EAST SIDE OVERFLOWS

Drainage Area No.	Loadings (lb) 0-30 min. BOD_5	TSS	30-60 min. BOD_5	TSS	60-90 min. BOD_5	TSS	90+ min. BOD_5	TSS
28	510	1960	1590	7000	730	3400	640	2450
29	1090	6940	2410	9290	600	2650	130	430
31	2650	3330	4760	8250	1070	2610	600	600

	Total Loadings (lb) BOD_5	TSS	Total Overflow (mil gal)
28	3470	14810	10.05
29	4230	19310	20.88
31	9080	14790	10.50
	16780	48910	41.43

TABLE 24. AVERAGE ANNUAL LOADING TO GENESEE RIVER SIMPLIFIED STORMWATER MODEL

Drainage Area No.	Overflow (mil gal)	BOD_5* (mg/l)	TSS* (mg/l)	BOD_5 (lb)	TSS (lb)
7	35	63	192	18390	56050
8,9	1370	85	171	971190	1953810
16	445	135	547	501030	2030080
21	51	45	74	19140	31480
22	277	284	496	656090	1145850
25	285	40	114	95080	270970
Front St.	26	30	89	6510	19300
Totals	2489			2267430	5507540
Avg Daily Loading:				6210	15090

*Note: These values represent average pollutant concentrations based on volumes given in Table 21 and concentrations given in Table 20.

Treatment Considerations

To further aid in assessing various abatement alternatives, analyses were conducted on the use of primary swirl concentrator units installed as site treatment facilities at consolidated overflow locations. The size, efficiency, and performance of the swirl units used for the analyses were determined from the pilot plant studies (8). Based upon work done at the LaSalle Hydraulic Institute and the pilot plant work conducted under this study there appears to be a design Froude number for optimum unit performance. For this study a Froude number of 0.167 was established. The design Froude number in conjunction with the established design overflow rate, defined as that flowrate occurring at the end of the first hour of overflow (previously defined as the first-flush), was used to develop the size of the primary swirl units for each overflow location. Table 25 shows the number of swirl units required for each site based on sizing requirements presented in Volume II of this report.

TABLE 25. NUMBER OF REQUIRED SWIRL UNITS BY DRAINAGE AREA

Drainage Area No.	Number of Units
7	4
8,9	6
16	4
21	3
22	3
25	3
Front St.	2
Siphon	2

The efficiency of the primary swirl unit is related to the influent solids concentration and the Froude number which is determined from the actual influent flowrate. Table 26 shows the overflow storage capacity required to capture the first-flush volume for the 2 Yr-2 Hr design storm.

These volumes were determined by application of the SWMM. The overflow in excess of the first-flush volume under this alternative was to be applied to the primary swirl units for solids removal. Based on the size and efficiency of each swirl unit, Table 27 indicates the amount of solids removed by the design swirl units treating post first-flush flows.

TABLE 26. OVERFLOW STORAGE CAPACITY REQUIRED TO RETAIN FIRST-FLUSH 2YR-2HR STORM

Drainage Area No.	Volume (mil gal)
7	3.91
8,9	12.89
16	6.00
21	2.43
22	4.57
25	5.08

TABLE 27. SWIRL UNIT SOLIDS REMOVAL EFFICIENCY OF FIRST-FLUSH 2YR-2HR STORM

Drainage Area No.	TSS Influent (lb)	TSS Removed (lb)	Efficiency (%)
7	4860	760	16
8,9	39450	11900	30
16	9500	6500	68
21	3370	0	0
22	3830	2430	63
25	4030	950	24
Front St.	4790	0	0
Siphon	29480	14480	49
Totals	99310	37020	
Avg. System Efficiency			0.37

Therefore, with capture of the first-flush and primary swirl unit treatment on the post first-flush flows, the total TSS load to the Genesee River would be reduced to 62,290 lb; that is, the swirl units remove an additional 37,020 lb of TSS. The removed solids could be pumped back into the St. Paul Boulevard Interceptor and conveyed to wet-weather treatment facilities at the Van Lare STP. In addition, the solids removed from the overflows from the east side of Rochester which discharge to the Cross-Irondequoit Tunnel and are conveyed to the Van Lare STP, must be determined. Following a similar procedure for calculating BOD$_5$ and TSS loadings associated with the River overflows, Table 28 presents the pollutant loadings and flows projected for several east side overflows discharging to the Tunnel.

TABLE 28. EAST SIDE POLLUTANT LOADINGS FOR 2YR-2HR STORM

Drainage Area No.	Flow (mil gal)		BOD$_5$ (lb)		TSS (lb)	
	F-F	Post F-F	F-F	Post F-F	F-F	Post F-F
28	5.85	4.20	2100	1370	8970	5850
29	17.59	3.29	3500	730	16230	3080
31	7.98	2.52	7410	1670	11580	3210
Sub-Totals	31.42	10.01	13010	3770	36770	12140
Totals	41.43		16780		48910	

SECTION 8

DESCRIPTION AND EVALUATION OF ALTERNATIVES

GENERAL

The 1972 Council of Environmental Quality Third Annual Report indicated that in 80 percent of the urban areas studied, downstream water quality was not controlled by point sources. The urban contribution to these water quality contraventions of defined standards is in the form of stormwater runoff and combined sewer overflow.

There are three general approaches available for the control of stormwater and combined sewer overflow discharges. A combination of storage and treatment can be applied through the use of structurally intensive measures to attain defined water quality standards. Secondly, the sources of the pollutants to the sewer system can be addressed through the application of the Best Management Practices (BMP) concept. Thirdly, a combination of the two can be applied. In general, the most cost-effective abatement alternative will be the application of structural and non-structural (BMP) measures.

In light of the very significant and substantial capital and operating costs associated with the implementation of structurally intensive alternatives, the application of Best Management Practices offers itself as a very attractive alternative to the solution of urban runoff problems. Optimized source and collection system management can provide a more quickly facilitated and less costly alternative for the abatement of wet-weather induced quality impairment of the receiving waters.

For this Report the alternatives that were evaluated can be classified as nonstructural, minimal structural, and structurally intensive. The first two classes of abatement alternatives are those generally considered in developing a BMP program.

NONSTRUCTURAL ALTERNATIVES

Land Use Policies

Land use policies and sewer system maintenance procedures can significantly affect both the quantity and quality of combined sewer overflows. Building, plumbing, and zoning codes directly influence the percent imperviousness and dry-weather flow characteristics of a drainage area.

Zoning ordinances are usually based on the concept of the best use of land, that is, the use that will most significantly benefit the community as a whole. Generally, increases in the relative imperviousness of an area adversely alter runoff and overflow characteristics; therefore zoning which limits development of impervious areas is beneficial from the aspect of stormwater management. Urban renewal initiated changes in zoning generally tend towards the development of open spaces or towards lower imperviousness areas. Citizen and commercial initiated changes are often opposite to those initiated by urban renewal, especially when land is bought on speculation for later development involving commercial shopping centers and apartment complexes. Therefore, because urban land has a high market value, zoning changes favoring open spaces, and thus stormwater management measures, are difficult to implement. There are, however, zoning alternatives that favor overflow reduction.

Planning boards and reviewing agencies may establish codes benefiting overflow reduction. Restrictive ordinances, which eliminate direct entry of sump pumps and roof drains into the sewer system, have the same effect as reducing the imperviousness in a drainage area because runoff and peak runoff entering the sewer are necessarily reduced. Specifying the use of porous pavement in those areas with suitable subsoil conditions is another method that also has the effect of increasing perviousness. Planned open space within developed areas, which can be used for both recreation and surface storage, is multifunctional and maximizes land use.

Street Cleaning Practices

Street cleaning in the City of Rochester includes the cleaning of any debris that may be found on the streets. This operation not only includes street sweeping but also the picking up of bulk material which may be left over by refuse collection and tree care operations.

The City of Rochester is divided into 42 residential and seven central business district cleaning routes. The normal street cleaning schedule for the residential routes is every sixth working day; that is, every street will be cleaned on both sides every sixth working day. This schedule is followed to avoid the alternate parking problem. The seven central business district routes are cleaned each working day between the hours of 5 AM and 1 PM. Scheduled cleaning operations may be delayed several days at times due to adverse weather conditions.

The vehicle used by the City of Rochester is an Elgin & Wayne Sweeper having an approximate cost of $40,000. The City owns 17 such vehicles. Only nine sweepers are, however, employed during normal operations. The efficiency of these sweepers depends to a large degree on the operator. According to City maintenance personnel estimated efficiencies during the spring months average between 60 and 70 percent while during the summer months the efficiencies may average between 80 and 90 percent. Efficiency of the street cleaning operation is also dependent on the number of cars parked on the street. Sweeper maintenance is very high; however, the average vehicle repair time is usually less than a day.

Immediately following the winter months, a major spring cleanup program is implemented for a period of six weekends. At this time, additional equipment from other divisions is used on weekends to help remove bulk materials from the street. During this period, the weekday sweeper operations are concentrated in those areas where bulk materials are removed. During spring cleanup, the cleaning of the central business district's routes is increased from five to six days per week, weather permitting. A program similar to spring cleanup is also implemented for a period of 8 to 10 weekends in the fall to remove leaf accumulations.

The estimated capacity of the Elgin & Wayne Sweeper is about 3 cu yd. Material that is picked up by the sweeper is taken to a transfer station where the material is transported to a sanitary landfill.

Presently, there are about 564 miles of street being cleaned in the City of Rochester, covering an area of about 36 sq mi. Cost estimates for the Rochester street cleaning operations are about $0.37 per foot of street per year or approximately 1.1 $ mil per year. This figure represents the total costs involved in the entire street cleaning operations for the City of Rochester Department of Public Works. Included therefore, are manpower, vehicle purchase, operation and maintenance, and hauling away of the collected solids.

Increased frequency and efficiency of street cleaning, up to a point, results in a noticeable decrease in the solids that accumulate during periods of no rainfall. However, increased volumes of solids must necessarily be processed and transported to a suitable landfill area. Also, there may be more toxic concentrations or amounts of leachates in the additional solids removed.

Although the SWMM quality routines did not simulate to any reasonable degree measured overflow pollutant concentrations, the benefit of increased street cleaning as predicted by the SWMM model is shown in Figures 30 and 31 for the Thomas Creek drainage area. Thus, the following discussion actually represents a sensitivity analysis of the quality portion of the SWMM. In the following seven figures CLFREQ refers to the number of days between street cleanings and land use classifications 1, 3 and 5 refer to single family residential, commercial, and park lands, respectively. Using the June 19, 1975 storm as a base, decreasing the street cleaning frequency from 5 to 25 days resulted in a corresponding increase in pollutant loading to the river of 52 percent for BOD_5 and 29 percent for TSS. For this simulation the number of sweeper passes has been set at one and the number of antecedent dry days has been set at 30. Antecedent dry days represent the total number of days when the cumulative rainfall is less than one inch of rain. On the average, a rainfall event occurs every four days in the Rochester area with an average ten day rainfall of about 0.9 inches. Therefore, the 30 day antecedent conditions represents an extreme situation, but previous model simulations indicate that little change occurs in runoff quality by increasing the number of dry days beyond 15 days which is about the number of days required to accumulate one inch of rain in Rochester.

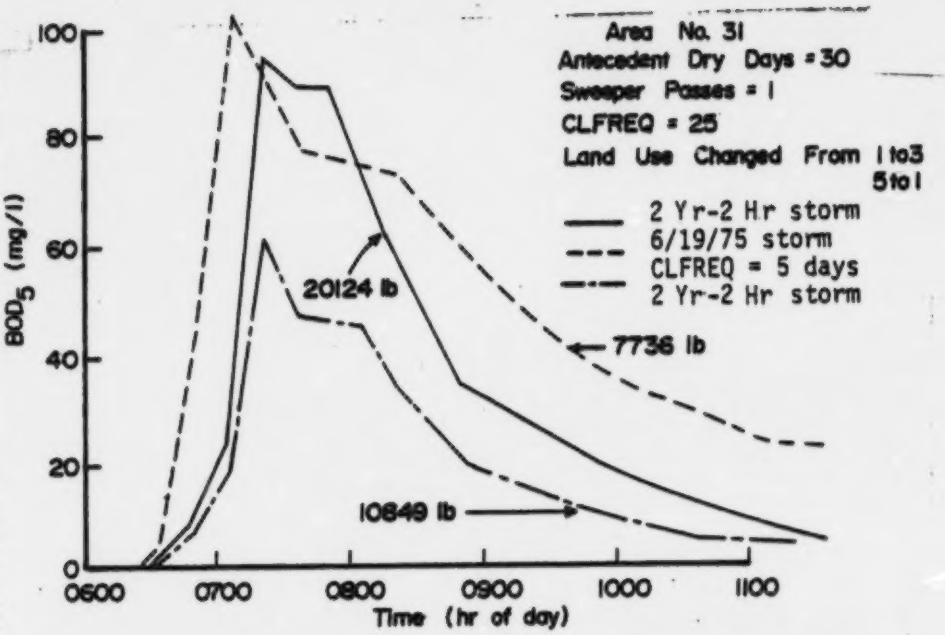


FIGURE 30. Runoff BOD5 Profiles for Various Storms and Street Cleaning Frequency

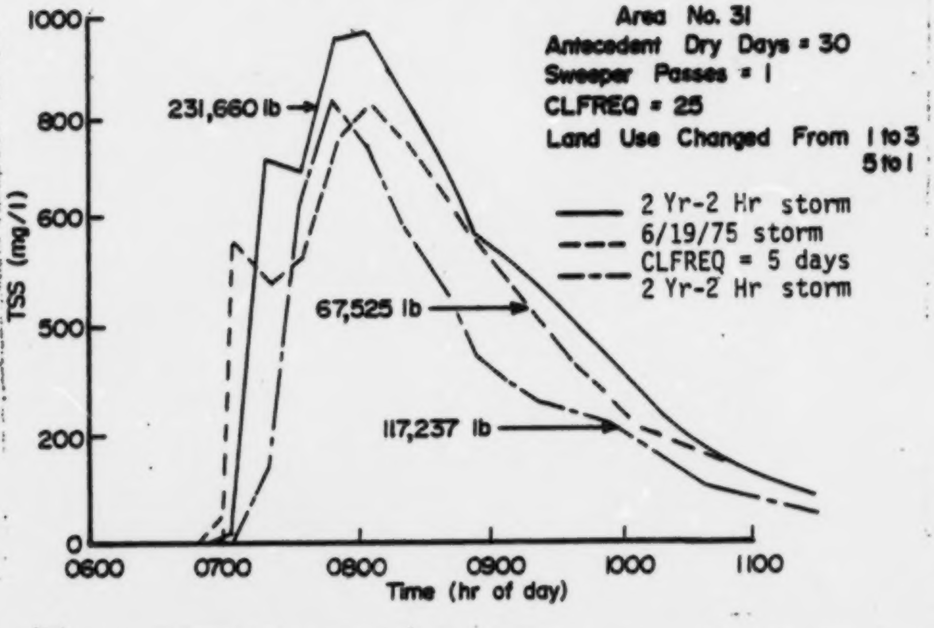


FIGURE 31. Runoff TSS Profiles for Various Storms and Street Cleaning Frequency

Figures 32 & 33 give a street cleaning comparison for drainage area no. 31 for both the June 19, 1975 storm and the 2 Yr-2 Hr storm. Figure 32 shows that the total overflow from area no. 31 is 14.3 mil gal for the June 19, 1975 storm and 39.5 mil gal for the two year storm. Cleaning frequency and efficiency have no effect on overflow quantities. The effect on overflow quality of increasing the cleaning frequency under the 2 Yr storm is similar to that projected under application of the June 19, 1975 storm; decreasing the frequency from 5 to 25 days results in a decrease of approximately 50 percent in BOD_5 and TSS loading from the combined sewer overflows. Despite the differences between storms (0.6 in./hr peak for the June 19, 1975 storm and 1.15 in./hr peak for the 2-Yr storm) the overflow concentrations for BOD_5 and TSS follow very closely in both magnitude and frequency when the cleaning frequency is 25 days. This observation is verified by the fact that the total BOD_5 and TSS loadings for the June 19, 1975 storm are in direct proportion to the 2-Yr storm loadings.

The efficiency of street sweeping depends not only on the frequency but also on the efficiency of the equipment used. From the previous discussion it would appear that the average efficiency is about 50 percent which is very similar to the default value applied in the SWMM for similar cleaning conditions. The effect of cleaning efficiency in reducing annual pollutant loading is not nearly as significant as cleaning frequency. The effectiveness of cleaning efficiency in improving overflow quality is not improved when the frequency is increased beyond the 5-day interval; (See Figure 34 which presents the initial time interval extraction of the runoff quality equation from the SWMM). In fact, it is seen that cleaning frequency is as important as dirt and dust build-up and total length of street to be cleaned in determining surface runoff quality.

In conclusion, the impact of street cleaning is significant. Because drainage area no. 31 is representative of all other areas in the city, it would appear that increasing the frequency of sweeping five-fold results in about a 50 percent reduction in BOD_5 and TSS loadings. Because the pollutant build-up is greater in commercial areas, these areas require more frequent street sweeping than residential areas (by about a factor of four) to maintain an equivalent overflow loading.

Note that the effect, as presented, of street sweeping operations on the quantity of BOD_5 and TSS discharged from the Rochester system has been predicted solely from the application of the SWMM. The analysis, however, does indicate a trend that can reasonably be expected. Actual reductions can only be determined through field demonstrations.

Increased Sewer Maintenance

Sewer maintenance has potential for not only reducing system flooding but also minimizing first-flush effects. The first-flush phenomenon observed from River overflows is presented in Table 9 and discussed at length in other sections. From that table, it is seen that nearly 65 percent of the total solids loading to the River occurs in the first hour when applying the 2 Yr-2 Hr storm. Similarly about 45 percent of

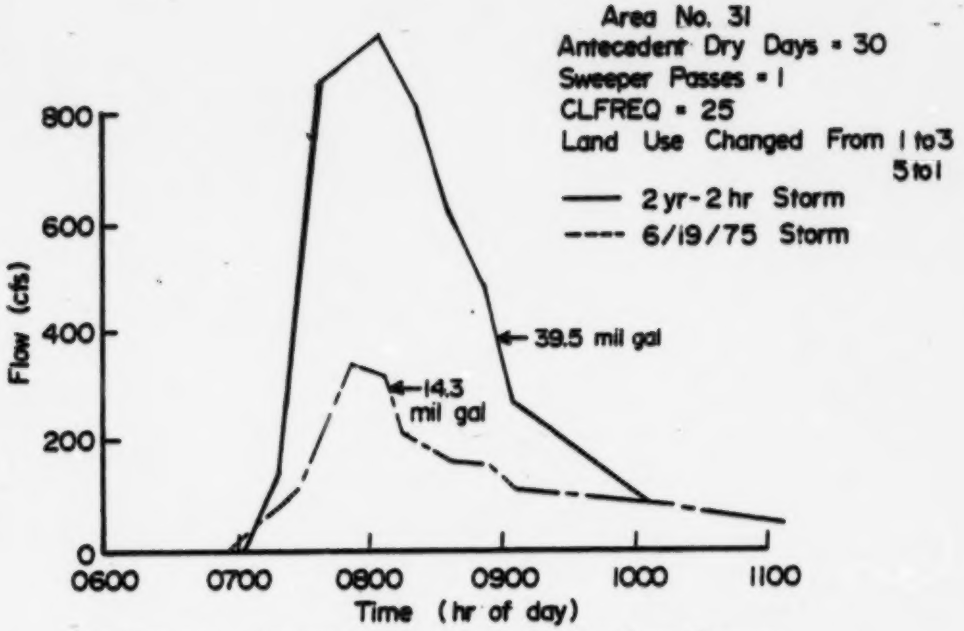


FIGURE 32. Runoff Hydrograph for Various Storms

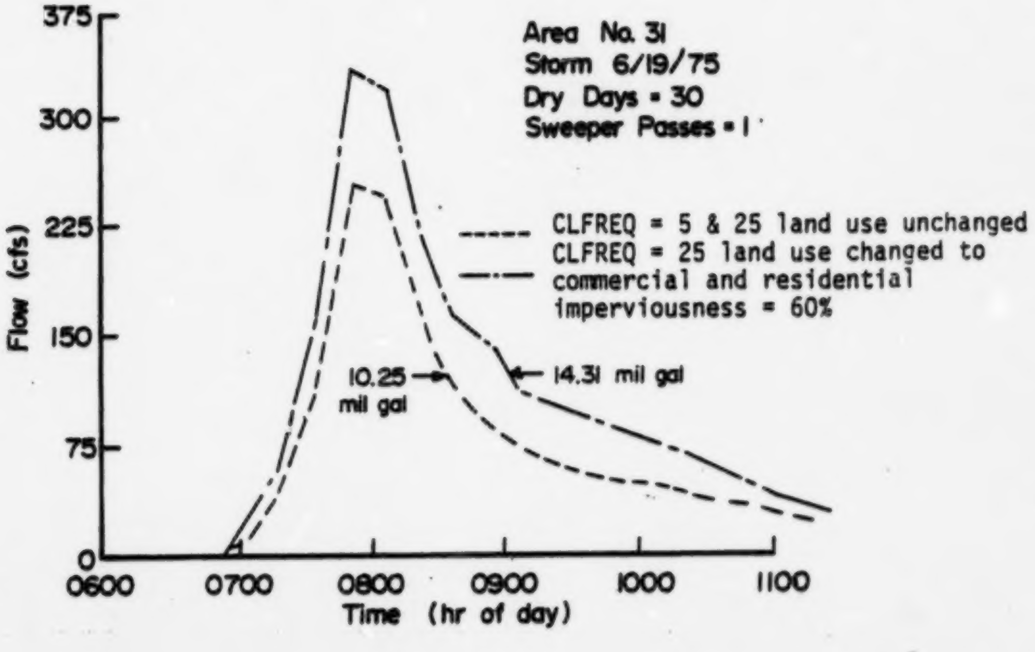


FIGURE 33. Runoff Hydrograph for Various Land Uses

$$
P_{shed} = \sum_{i\,=\,1}^{\text{No. of Subcatchments}} \left[\begin{array}{l} \mathrm{QFACT} * \mathrm{DDFACT} * \mathrm{GQLEN} * \underbrace{\left[\mathrm{CLFREQ} + \quad 1 + (1\text{-}\mathrm{REFF}) + \ldots + (1 - \mathrm{REFF})^{\frac{\mathrm{DRYDAY}}{\mathrm{CLFREQ}}}\right]}_{\boxed{\text{STREET SWEEPING}}} * 453 \\ * \underbrace{\left[.06 + 3600 * \frac{\mathrm{AVFLOW}}{\mathrm{WAREA}} * 12^{\;1.1} * 1.4 \right]}_{\boxed{\text{SUSPENDED SOLIDS AVAILABILITY FACTOR}}} * \left[1\text{-}e^{-(4.6 * \mathrm{AVFLOW}/_{\mathrm{WAREA}}\;\; 12 * \mathrm{DELT})} \right] \\ + \underbrace{\left[\mathrm{CBFACT} * \mathrm{CBVOL} * \mathrm{BASINS} * 28.3\right]}_{\boxed{\text{CATCH BASIN B.O.D. CONTRIBUTION}}} * \left[1 - e^{-(\mathrm{AVFLOW} * \mathrm{DELT}/1.6 * \mathrm{CBVOL} * \mathrm{BASINS})} \right] \end{array} \right]
$$

Note: Equation applicable for initial time-step only

DDFACT = Dirt and Dust Buildup rate (lb/Day/100 ft of Curb)

QFACT = Percentage of Dirt and Dust which is TSS or BOD_5

GQLEM = Curb Length (100 ft)

CLFREQ = Cleaning Frequency (Days)

REFF = Cleaning Efficiency (%)

DRYDAY = Number of Days where accumulative rain $\leq$ 1.0 in.
P_{shed} = Pounds of Pollutant Washed Off Land Surface During Runoff

FIGURE 34. SWMM Runoff Quality Equation

the total BOD_5 loading occurs in the first 60 min of the overflow event; however, in terms of volume of overflow, the first 60 min represents only about 37 percent of the total River overflow. The runoff effect for the 2-Yr storm is evident for about 10 hours; therefore the disproportionally high initial solids loading is probably due to solids deposited in the interceptor during the antecedent dry days. Periodic flushing of interceptors where dry-weather flow is less than 3 fps would therefore help to decrease the effect of first-flush by scouring solids under controlled conditions.

Data documenting major sewer complaints before and after a major cleaning program have been compiled in the City of Rochester and are shown in Table 29. The cleaning program conducted to date consisted primarily of sewer flushing but also included catchbasin cleaning. As indicated, the major identified sewer complaints were those associated with flooding problems. For the drainage areas in which cleaning occurred, the complaint calls were down by about 25 percent; for those in areas which cleaning did not occur, the total complaint calls were down by less than 10 percent for the same study period. Although insufficient data is available to say that sewer cleaning conclusively reduces flooding, it can be said that intensive sewer cleaning during the period of March 1975 through April 1976 had a significant impact on reducing local flooding problems, particularly since no major sewer network changes were made during this period.

TABLE 29 INFLUENCE OF SEWER MAINTENANCE

Drainage Area No.	No. of Major Sewer+ Complaints Before Sewer Cleaning Program	No. of Major Sewer Complaints After Sewer Cleaning Program
6	11	9
7	7	6
9	47	30
10	5	3
21	7	6
31	19	22
Totals	96	76

Study Period: March 1975 - April 1976

+ Each major sewer complaint is assumed to constitute a flooding problem

Land Use Planning

As previously discussed, increased imperviousness adversely affects the quantity and quality of stormwater and combined sewer overflow. Figures 35 and 36 show this impact. For the same cleaning frequency and associated dry days, increasing the land use in drainage area no. 31 from its present residential zoning (relative imperviousness equals 40 percent) to commercial and residential (imperviousness equals 60 percent) increases the total runoff by nearly 50 percent and increases the total overflow loading by

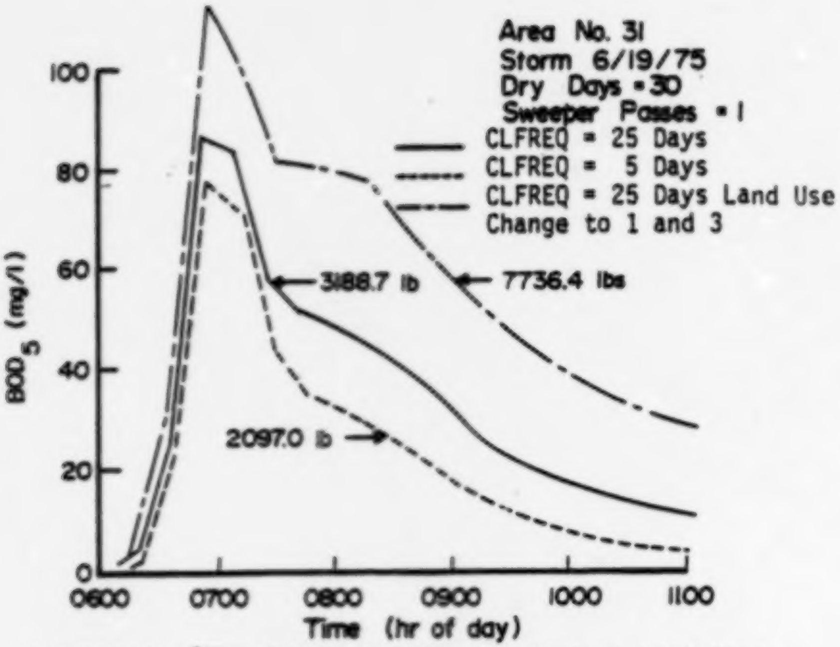


FIGURE 35. Runoff BOD_5 Profile for Various Land Uses

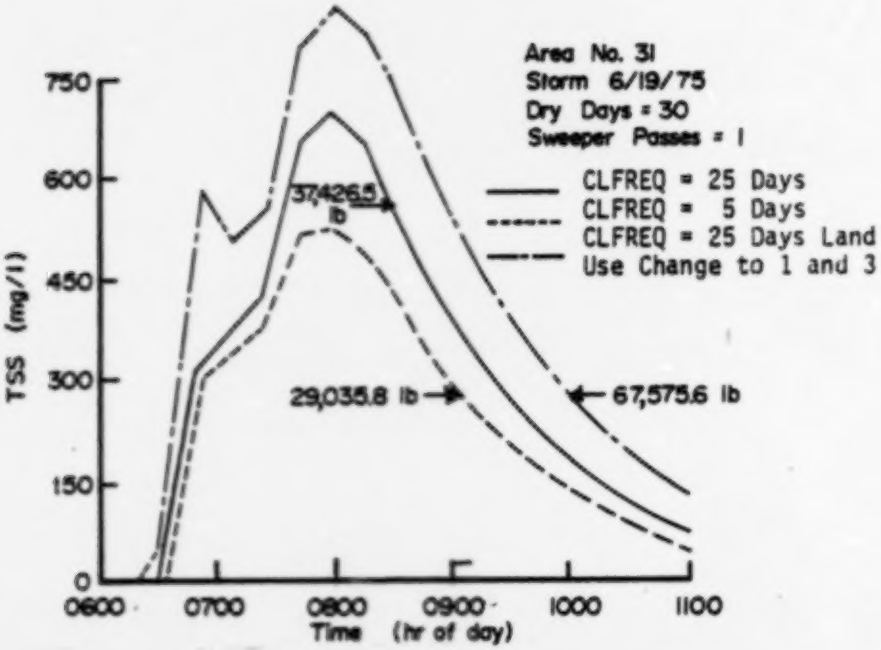


FIGURE 36. Runoff TSS Profile for Various Land Uses

85 percent for the 6/19/75 storm. Because loadings for the 6/19/75 and the 2 Yr design storm follow in nearly direct proportion to the ratio of overflows, similar percentage increases in overflow volume and loadings would occur using a 2 Yr storm for analysis. Restrictive zoning, use of porous pavements, and eliminating direct inflow from building roof and footing drains are effective land-use practices which are countermeasures to increased imperviousness.

One option available to planning boards and reviewing agencies is to require that each land use be associated with a certain percentage of open green areas and/or certain relative imperviousness values. Legislating green area development is easier than relative perviousness because the former has a visible impact on the public's aesthetic awareness. The latter, because of technological consideration, is more difficult to legislate; therefore, building codes should be established or altered to act as guidelines for attaining relative imperviousness values.

Building and plumbing codes are valuable and necessary tools which potentially have a great impact on alleviating and attenuating stormwater overflows. Enacting ordinances requiring porous pavement in parking lots and driveways could virtually eliminate runoff from impervious nonroad areas especially if roof drains are removed from the stormwater system and the runoff placed directly on the driveways and/or parking lots. Previous work (18) involving porous pavement indicates that commercial parking lots and driveways would require about 5 in. of porous pavement. In Rochester, a subbase of 2 ft would be required to prevent freeze-thaw damage to pavements. This configuration of porous pavement and subbase gives a storage capacity of about 10 in. of rain which is far in excess of the 2 Yr-2 Hr design storm total rainfall of 1.15 in. Because the roof area to impervious area ratio is about 0.5 for commercial areas, the total rainfall that could be put on parking lots and driveways would be 2.3 in. for a 2 Yr storm providing roof drains were removed from the sewer system. Furthermore, the porous pavement has the capacity to store far more than this 2.3 in., since the average 10-day rainfall in Rochester is about 0.9 in., and the soil required for percolation is a fine sand and silt mixture which would require no subdrains. Rochester soils are highly variable and consist of glacial drift, shale, and limestone that have a much higher permeability than silt and sand. Therefore, the use of porous pavement in conjunction with separtion of roof drains from sewers could virtually eliminate runoff tributary to combined sewer systems from commercial and industrial areas which constitute about one-fourth of the total city area.

Residential areas amount to one-third of the city area and have a relative imperviousness of about 40 percent with a roof area to total impervious area ratio of about 35 percent. Removing roof drains from sewers in this area would not eliminate runoff from roof areas but would attenuate this runoff; therefore, the imperviousness of the area would effectively be reduced to a value between 25 and 40 percent. Using porous pavement for driveways and placing roof drainage on these driveways would reduce the percent imperviousness in residential areas to about 15 percent which is similar to the area presently represented by the streets. The annual effect of imperviousness on total annual overflow in drainage area no. 31 is shown in Figure 37.

Present land use results in an annual overflow of about 22 mil gal while removing roof drains would decrease overflows by 14 mil gal for the same land use. Commercialization of drainage area no. 31 using present building and plumbing codes would result in an annual overflow from this area of almost 100 mil gal. Using porous pavement and discharging roof drainage on this pavement in commercial parking lots and driveways could result, under specific applications, in an annual overflow reduction of about 90 percent.

Surface Storage

Surface storage is a very useful tool in relieving runoff surges that would otherwise surcharge storm and/or combined sewers. Because of the high value of land in urban areas and because of the large areas required, implementing this concept is extremely difficult. Using available land such as parking lots, roof tops and open spaces for flow attenuation as opposed to construction of specific stormwater retention ponds would appear to be more feasible. Storing the five-year storm on roof tops would amount to an increased stress of about 9 psf which is well below the 40 psf snow design loads in this region. Roof storage is only feasible in commercial and industrial areas and would result in an approximate total runoff reduction of 10 percent. Open areas and park land contribute about 20 percent of the total surface area. Flooding park land and open spaces appears to be more feasible than roof storage with respect to total runoff capture, but the disadvantage to flooding is post-storm grit and debris clean-up. Because of this disadvantage, flooding parkland does not appear to be feasible. Storage ponds are required to effectively handle runoff surges and thus reduce overflows. Large land areas, as well as separation of sanitary and storm sewers, are required for depression storage and therefore its application is significantly limited in developed areas. In conclusion, surface storage in the urban areas characteristic of Rochester would be extremely difficult to implement and therefore is not considered practical.

Each of the nonstructural CSO abatement alternatives has been evaluated relative to the Rochester study area. The drainage areas identified as being most suitable for the application of these source management measures are presented in Figure 38. Increased street cleaning would be most effective in those areas that receive the highest daily loadings such as downtown urbanized areas. Area no. 25 represents the heavily trafficked downtown portion of the City of Rochester. Area no. 7 represents another area that receives heavy traffic and thus is subjected to substantial surface pollutant accumulations. Eastman Kodak Company has several large industrial plants within this area that employ many hundreds of people. Thousands of cars and trucks use the main routes through this area daily. Area no. 16 represents an area essentially undergoing urban renewal. Much of the area is presently in poor physical condition and large amounts of debris litter the entire drainage basin. Area no. 22 represents another section undergoing urban renewal which is presently in poor condition with

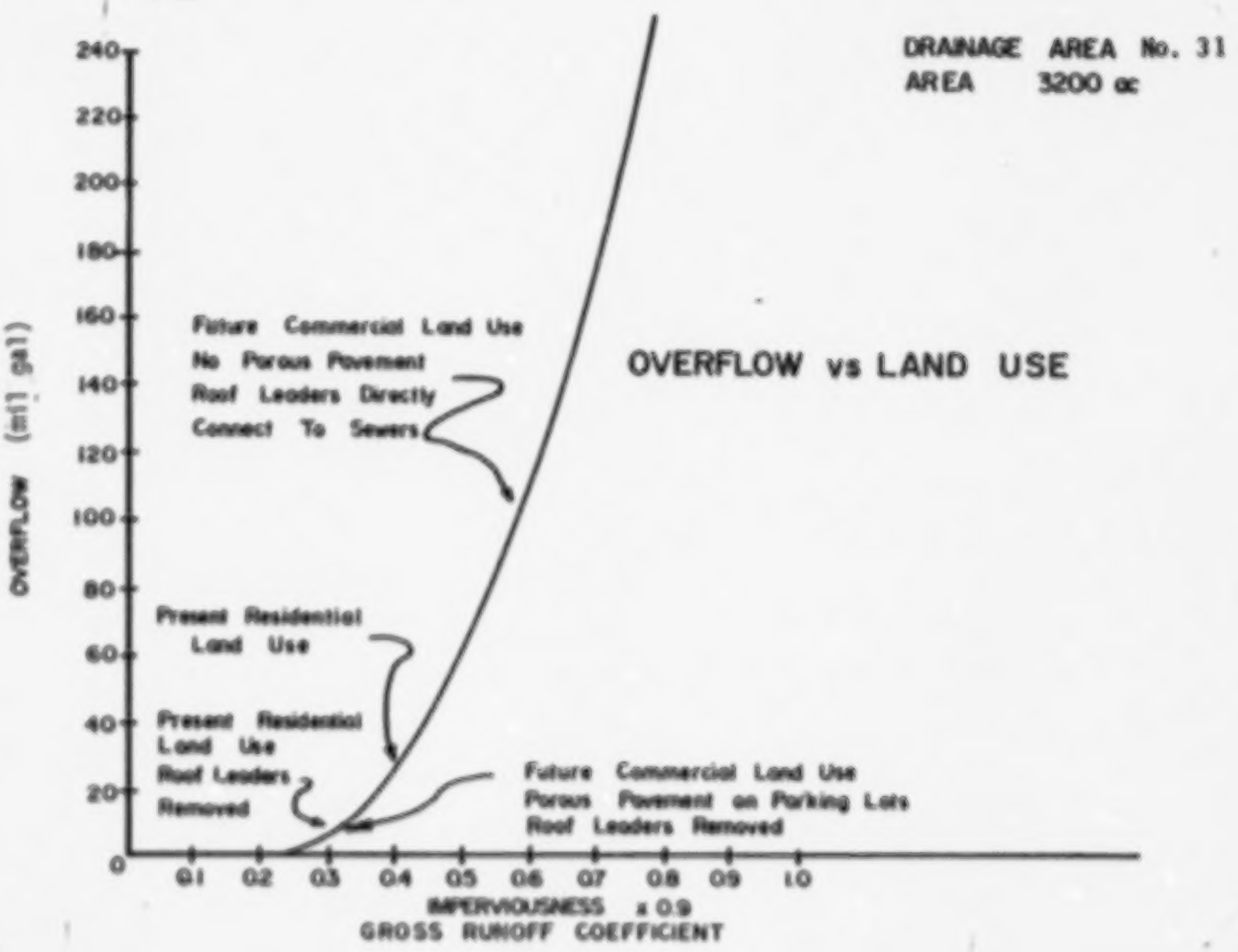
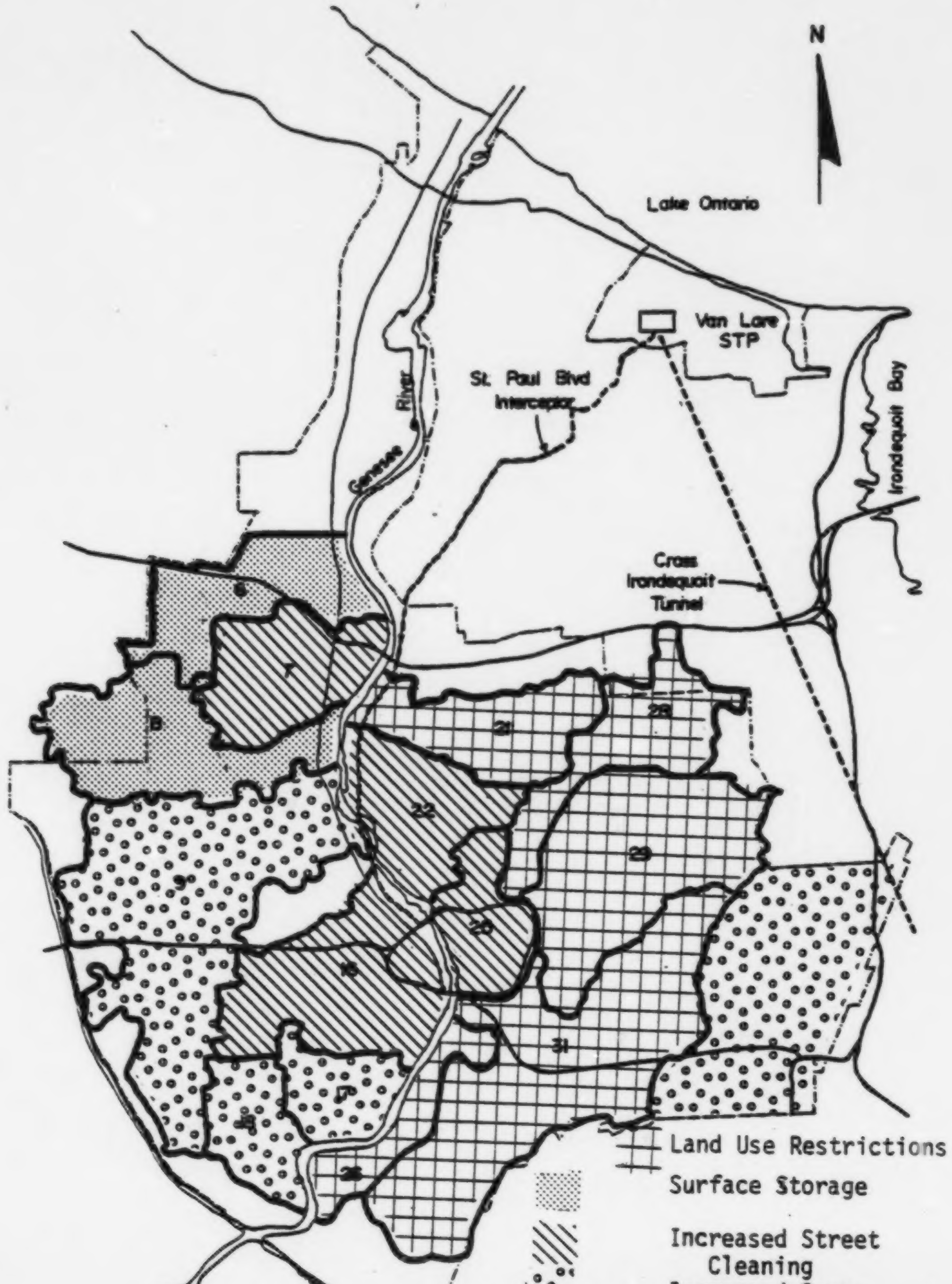


FIGURE 37. Overflow Volume as a Function of Land Use


N
Lake Ontario
Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
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Land Use Restrictions
Surface Storage
Increased Street Cleaning
Increased Sewer Maintenance

FIGURE 38. Location of Drainage Areas Suitable for the Application of Nonstructural Alternatives

a large volume of surface contaminants.

Relative to increased sewer maintenance, Areas nos. 16, 17 and 18 have been previously identified as those areas with sewers requiring cleaning and repair. Such repairs will improve the conveyance capacities of these sewers which can reduce the frequency and volume of combined sewer overflow.

Areas nos.6 and 8 represent areas that are essentially entirely all commercial and industrial. With the large number of parking lots and roof tops in these areas, implementation of surface retention practices such as ponding on roofs and in parking lots can relieve the hydraulic load imposed on the downstream sewers, that in turn can also reduce the frequency and volume of CSO.

Areas nos.21, 16, 28, 29 and 31 are essentially residential in nature. Implementation of varying land use restrictions such as no additional pavement or buildings can possibly relieve the hydraulic load on the East Side Trunk Sewer thereby reducing CSO occurrences presently experienced along this major conveyance system.

MINIMAL STRUCTURAL ALTERNATIVES

Abandoned Treatment Facilities

Within the study area there are only two possible abandoned treatment plant or chlorination facilities for use in the abatement of combined sewer overflow. One is the Joseph Ward Chlorination Station located near the intersection of Joseph Avenue and Ward Street. The station has not operated since its construction in 1965. Because the drainage area tributary to this station is rather small and it is relatively remote from system overflows, it is doubtful whether this facility could be effectively used in the treatment of such overflows.

The other possible facility is the Norton-Densmore Chlorination Station located on the north side of Norton Street at the junction with Densmore Creek. This combination screening and chlorination station was constructed to treat the combined sewer overflow from the Norton Street Tunnel. In addition to the overflow from drainage area no. 28 conveyed by the Norton Street Tunnel, overflow from drainage area no. 29 also discharges to the Densmore Creek location. Presently, these two overflows discharge to Irondequoit Bay via an open channel. Under construction, approximately 1000 ft downstream of this station is a dropshaft into the Cross Irondequoit Tunnel. Use of this facility for combined sewer overflow pollution abatement could involve screening and chlorination of the overflow prior to discharge to the tunnel.

The expected peak flow rate through the station resulting from the 2 Yr-2 Hr design storm is approximately 1800 cfs. Several problems become apparent when considering such a large flow. It is advisable to remove a large percentage of the organics prior to chlorination to substantially lower the detention time necessary for coliform kill. Even with high-rate treatment facilities, large basins would be required for solids removal for such large flowrates. Space limitations at this site preclude the use of any large basins or tanks.

Use of a swirl concentrator at the station is not practicable because of size requirements. Based on information obtained from the pilot plant studies, the primary swirl unit would have to be approximately 250 ft in diameter to adequately treat the peak flowrate.

The problems associated with both the Joseph Ward and the Norton-Densmore Chlorination Stations preclude their use as facilities for combined sewer overflow abatement.

Multi-Purpose Upstream Impoundment for Overflow Control

One method for this alternative is underground overflow volume storage for subsequent release after the storm event. Depending on invert and ground elevations, the stored volume may be emptied by gravity or may have to be pumped.

On an annual basis, as predicted by the Simplified Stormwater Model, the Maplewood Park drainage area no. 7 contributes only two percent of the total river overflow. Based on measured time-incremented quality data, it is estimated that area no. 7 contributes one percent of the total average annual loading of BOD_5 and TSS. Although overflow from this area does not represent a substantial pollutant load to the river, it is used as an example to illustrate the procedure to be followed in determining the size of facility necessary for upstream impoundment.

Sizing for underground storage must be based on a defined quantity of wastewater. To remain consistent with the other alternatives the 2 Yr-2 Hr design storm is used to evaluate the effectiveness of upstream impoundment for overflow control. Overflow from drainage area no. 7 for this storm event amounts to 9 mil gal. To capture this volume of overflow completely, an underground storage chamber having dimensions of 300 x 300 x 15 ft would be necessary. The chamber could be constructed under the park immediately before the overflow discharge outlet to the river. The two weir overflows located within the drainage area would continue to function normally. Overflows would occur but would be intercepted by the detention chamber before the outlet to the river. Volumes of overflow generated from storms greater than a two-year event would be allowed to bypass the detention chamber. The stored volume would be detained until the termination of the storm event and then pumped back into the interceptor system at a controlled rate.

The described overflow detention chamber could also provide for equalization of dry-weather flow. During periods of excessively high dry-weather flow, the wastewater could be temporarily detained and then pumped back into the system. This would lessen the load on the sewer system and also would eliminate the possibility of dry-weather overflow or overloading of the dry-weather treatment facility.

Similar considerations for the other drainage areas indicate the possibility of underground storage chambers for overflow pollution abatement. Several problems, however, would be encountered with such structures. One is the sludge or solids removal problem associated with all types of detention structures. The other is the poor subsurface conditions encountered throughout the Rochester area.

Considering solids removal, it is estimated that 5500 lb of solids are generated from the analyzed Maplewood detention chamber. It is not practicable to pump these solids back into the interceptor system because of the possibility of settling in the sewers during the reduced flowrates after the storm. Therefore, it would be necessary to remove the bulk of these sludges after the storm event. Additional study with emphasis on local soil conditions must be conducted to further evaluate this alternative.

Use of In-System Regulators and Control Structures for Increased In-System Storage

The use of inflatable dams for increased in-system storage can be effective in reducing the number and volume of combined sewer overflows. This is especially true considering the number of large tunnels that are proposed or under construction within the Rochester Pure Waters District. The Genesee Valley Interceptor (GVI) currently under construction has a storage volume of approximately 25 mil gal. This interceptor provides combined sewage drainage relief for the areas presently served by the Genesee Valley Canal Sewer and will eliminate all combined sewer overflows along the west side of the Genesee River south of the Upper Falls. The first overflow relief point is at Front Street. Assuming that the runoff coefficient is the same over the remaining portion of the drainage area tributary to the GVI as for areas nos. 17 and 18, the total volume of runoff from the 2 Yr-2 Hr design storm in 10.2 mil gal. Unless a control device is installed to limit flow from the interceptor, the available storage capacity will never be fully utilized. With the use of an inflatable dam, this interceptor can store as much as a 5-year storm with an associated total runoff of 14.5 mil gal. Average dry-weather flow accounts for an estimated 5.8 mil gal. The stored wastewater can be released into the Clarissa Street Tunnel at a rate of about 25 cfs without overloading any downstream facilities. At this rate the interceptor would drain in approximately 32 hr.

Analysis of the proposed Genesee River Southeast Interceptor indicates the need for some type of control device at the outlet if the large volume of available storage is to be utilized. This interceptor is to involve approximately 3 miles of 14 ft dia tunnel. The available storage capacity is estimated to be 17.3 mil gal. The interceptor relief overflow point on this system will again be at Front Street. The SWMM estimates surface runoff from this drainage area resulting from the 2 Yr-2 Hr design storm to be 21.5 mil gal. Based on the storage capacity and the runoff volume a relief discharge rate of approximately 52 cfs is required to prevent surcharging the interceptor. At this rate the interceptor would drain in approximately 14 hr following the runoff event.

Storage capacity is also available in the Cross Irondequoit Tunnel and the proposed Culver-Goodman Tunnel complex. The control device necessary for storage and relief discharge will be the pump station located at the outlet of the Cross Irondequoit Tunnel before discharge to the Van Lare STP. The number of pumps, pumping capacity, and pumping duration will determine the extent of storage that is actually used. The Cross-Irondequoit Tunnel and the proposed Culver-Goodman Tunnel have a total storage capacity of approximately 85 mil gal. This is sufficient to completely capture all combined sewer overflows from the East Side Trunk Sewer for the 5 year design storm (66 mil gal). These overflows originate from drainage areas nos. 28, 29, and 31.

Regarding the present sewer network, there is insufficient storage capacity available in the conveyance sewer system of any drainage area to make the use of inflatable dams or other in-system control structures feasible for overflow control. Figure 39 identifies system locations where inflatable dams and/or regulators can be utilized to enhance upstream storage. Also identified are areas offering potential for minimal surface and subsurface storage.

Upgarding the Existing St. Paul Interceptor

The present St. Paul Interceptor system contains three major flow constrictions, one being the lower reach of the interceptor near the Van Lare STP, another downstream of inlets from drainage areas nos. 8, 9, and 22, and the last being the River Siphon itself. The location of the flow constrictions are presented in Figure 40. The first constraint near the STP consists of three interceptor sections ranging in unsurcharged capacity from 168 to 260 cfs. Closest to the Van Lare STP is approximately 240 ft of 5 ft-6 in. pipe having a capacity of nearly 260 cfs, and finally a section of the same diameter pipe about 1800 ft in length and having a capacity of 160 cfs. The second constraint is a 5 ft-4 in. x 5 ft-10 in. brick conduit directly downstream of the interceptor inlets from drainage areas nos. 8, 9 and 22. The third restriction, the River Siphon, consists of two cast iron pipes, 24 and 42 in. in dia, each approximately 3400 ft in length. The combined carrying capacity for the Siphon is slightly more than 70 cfs. Removing these constraints involves increasing the capacities to the unsurcharged capacity of neighboring conduits. The first constrained reach is increased to 285 cfs, the second to 250 cfs and the third (Siphon) to 140 cfs.

Table 30 which presents a tabulation of River overflows on an annual basis using the SSM would seemingly indicate that removing the constraints in the St. Paul Interceptor would play a major role in reducing the volume of combined sewer overflows as well as the frequency of overflow occurrences.

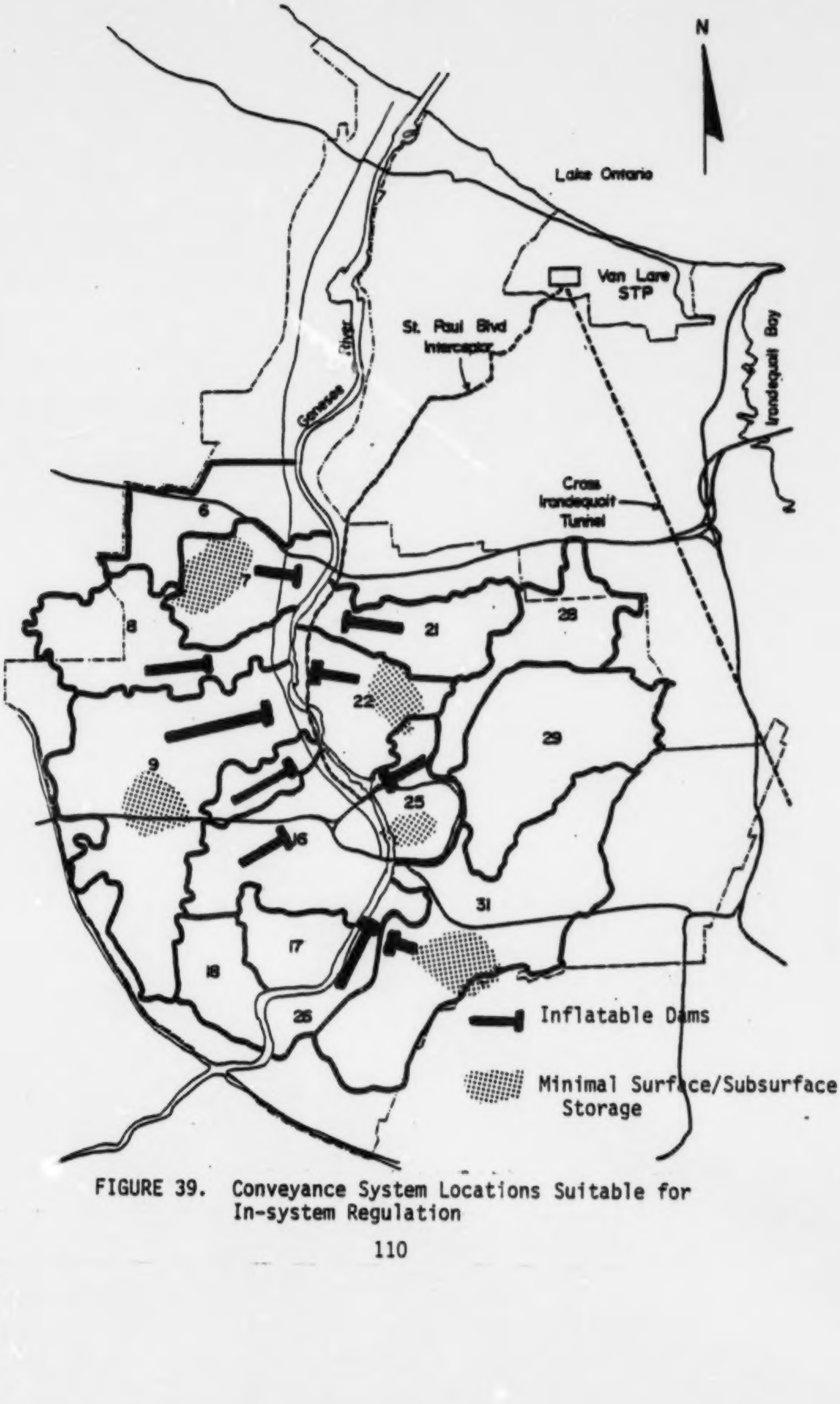

N
Lake Ontario
Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
6
7
8
21
28
22
29
9
25
16
31
17
18
26
Inflatable Dams
Minimal Surface/Subsurface Storage

FIGURE 39. Conveyance System Locations Suitable for In-system Regulation

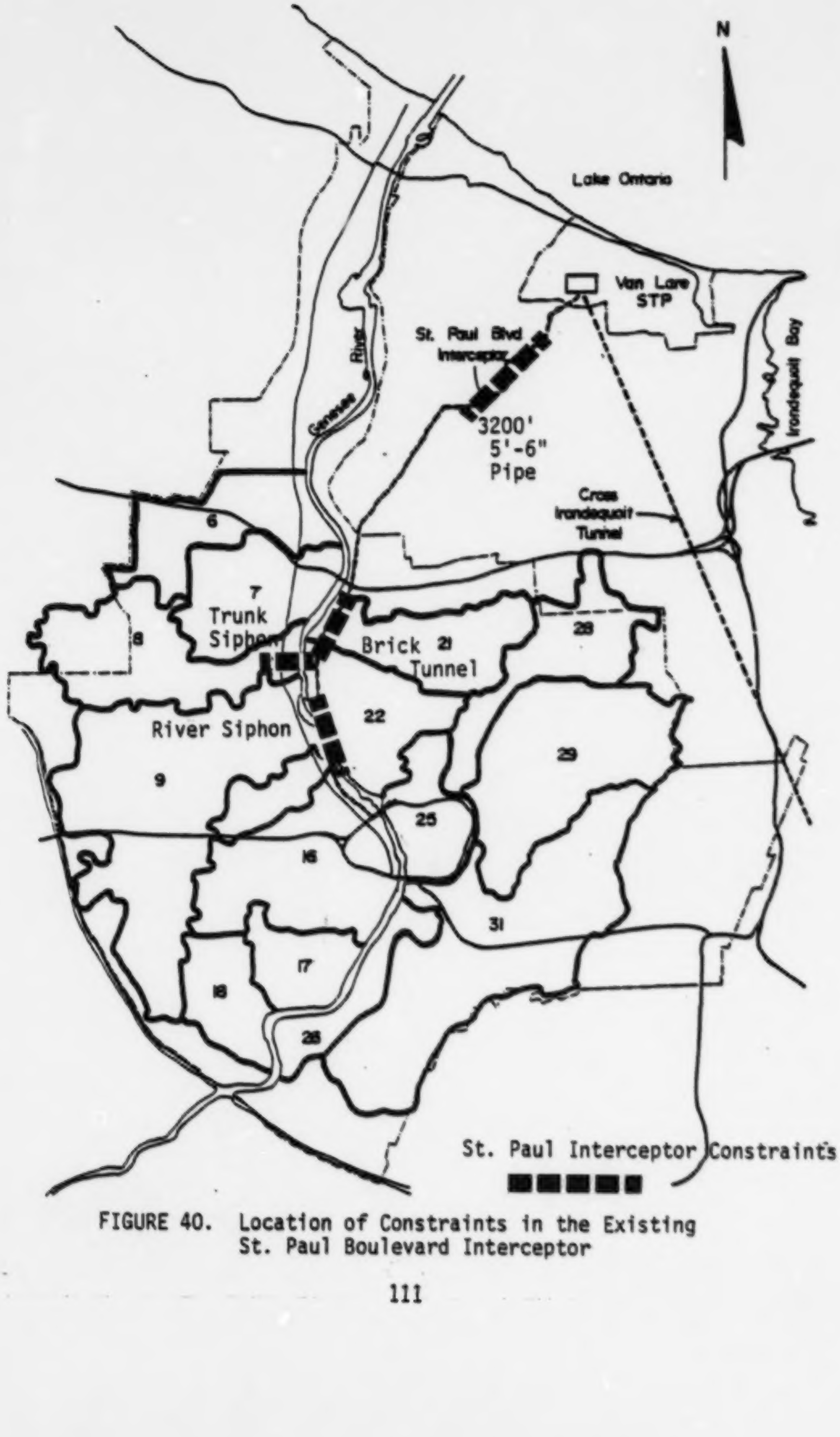

N
Lake Ontario
Van Lare STP
St. Paul Blvd Interceptor
3200' 5'-6" Pipe
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
6
7
Trunk Siphon
8
Brick Tunnel
21
28
22
River Siphon
9
29
25
16
31
17
18
26
St. Paul Interceptor Constraints
■■■■■

FIGURE 40. Location of Constraints in the Existing St. Paul Boulevard Interceptor

TABLE 30. RIVER OVERFLOW PROJECTED BY SIMPLIFIED STORMWATER MODEL FOR THE PROPOSED TUNNEL INTERCEPTOR SYSTEM

	Drainage Area No.	Average Annual Overflow (mil gal)	Average Annual Overflow (days)	% of Total River Overflow
St. Paul Interceptor Constraints Unaltered				
	7	22.4	7.55	1.4
	8 & 9	855.9	51.73	55.0
	16	278.0	62.05	17.9
	21	32.6	10.65	2.0
	22	173.3	56.25	11.1
	25	178.2	65.55	11.5
	Front St.	15.9		
	Total	1556.3		
St. Paul Interceptor Constraints Removed				
	7	22.4	7.55	5.3
	8 & 9	392.1	28.60	92.4
	16	0.0	0.0	0.0
	21	0.0	0.0	0.0
	22	0.0	0.0	0.0
	25	9.5	3.90	2.2
	Front St.	0.2		
	Total	424.2		

Removal of constraints in the St. Paul Interceptor are projected to reduce the annual total River overflow by 73 percent. Essentially a 100 percent overflow reduction occurs from drainage areas nos. 16, 21, 25, and Front Street with a 54 percent reduction in overflow from drainage areas nos. 8 and 9. Drainage area no. 7 is unaffected by these improvements. A similar analysis using the SWMM under the application of the 2 Yr-2 Hr design storm indicates an approximate 5 percent reduction in total overflow volume when the St. Paul Interceptor constraints are removed. This is illustrated in Figure 41 and Table 31.

The SWMM also indicates that removing constraints in the St. Paul Interceptor sufficiently lowers the hydraulic grade line at the regulators to effectively reduce River overflow. This action would reduce surcharging of the relatively low weirs, especially in drainage areas nos. 16, 21, 22 and 25 where the flows are relatively small and where the annual overflow is essentially eliminated. However, the analysis utilizing the 2 Yr storm does not project the same level of improvement. This discrepancy between yearly analysis and design storm analysis is understandable when one considers that 97 percent of all storms recorded in the twenty-five

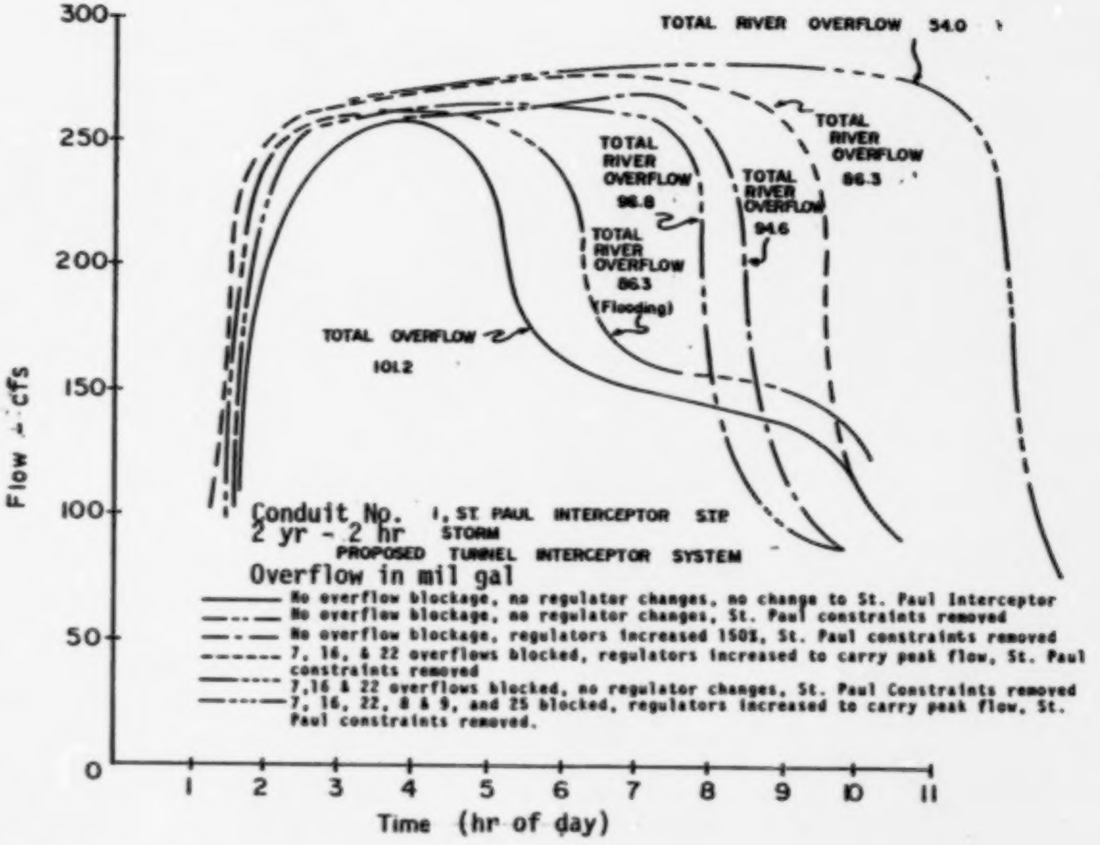


FIGURE 41. Minimal Structural Alternative Modeling Results (SWMM) Overflow Hydrograph

years of record had a total rainfall less than the 2 Yr-2 Hr design storm and 87 percent of all storms had a longer duration. On the annual basis, nearly every storm produces less total runoff than the 2 Yr storm and distributes that runoff to the interceptor system over a greater time period, thus reducing peak flows and subsequent overflows. The net effect of reducing the St. Paul Interceptor hydraulic gradient is therefore much more significant on an annual basis than what is indicated by the analysis of the 2 Yr-2 Hr design storm.

TABLE 31. EFFECT OF UPGRADING THE ST. PAUL INTERCEPTOR 2YR-2HR DESIGN STORM

	Present System Constraints Not Removed	Present System Constraints Removed	Proposed Tunnel Interceptor System Constraints Not Removed	Proposed Tunnel Interceptor System Constraints Removed
River Overflow (mil gal)	94.9	93.6	101.6	96.8
East Side Overflow (mil gal)	105.1	97.6	41.4	41.2
Total Overflow (mil gal)	200.0	191.2	143.0	138.0
Percent Reduction in Total Overflow With Constraints Removed	-	4.4	-	-
Percent Reduction in River Overflow with Constraints Removed	-	1.2	-	4.7

The overflow results obtained using the SSM were combined with the annual geometric mean overflow pollutant concentrations to give the annual river pollutant mass loadings. BOD_5 and TSS results are show in Table 32. It is interesting to note the magnitude of both BOD_5 and TSS loadings. The total loading projected from this analysis shows more than 2.5 mil lb of suspended solids and over 1 mil lb of BOD_5 entering the Genesee River annually. Comparison of both flow and quality data indicates that the 2 Yr-2 Hr design storm contributes about 6.5 percent of the total annual loadings.

TABLE 32. RIVER LOADING- PROPOSED TUNNEL INTERCEPTOR SYSTEM-PROJECTED BY SIMPLIFIED STORMWATER MODEL

	Drainage Area No.	Annual BOD_5 River Loading (lb)	Annual TSS River Loading (lb)
St. Paul Interceptor Constraints Unaltered			
	7	10,600	24,000
	8 & 9	550,000	1,220,000
	16	151,000	633,000
	21	13,300	20,000
	22	332,000	550,000
	25	59,000	169,000
	Front St.	4,000	12,000
		1,119,900	2,628,000
St. Paul Interceptor Constraints Removed			
	7	10,600	24,000
	8 & 9	252,000	559,000
	16	0	0
	21	0	0
	22	0	0
	25	3,340	9,500
	Front St.	40	130
		265,980	592,630

In summary, the annual analysis conducted with the SSM does not consider surcharge and therefore tends to underestimate the reduction in overflow volumes. The annual analysis uses daily rainfall data as opposed to hourly data. Previous work has shown projections made using daily rainfall data to underestimate the overflow volumes projected using hourly data by about 40 percent. These cancel one another and therefore the SSM reasonably predicts hydraulic flow conditions along the St. Paul Interceptor.

Selective Blockage of High Impacting Combined Sewer Overflows

Completely removing an overflow from the drainage system can be accomplished basically by two methods. One method completely captures the overflow using another collection sewer. The method hydraulically eliminates the overflow by properly sizing the existing interceptor, regulators, and weirs to effectively attenuate or remove peak flows. This method will be discussed here for selected overflows.

Eliminating overflows from drainage areas nos. 7, 16, and 22 would theoretically reduce total River overflow by approximately 25 percent which in turn reduces BOD_5 and TSS by about 35 percent for the 2 Yr-2 Hr storm. Likewise, eliminating overflows from drainage areas nos. 8, 9 and 25 in addition to those from areas nos. 7, 16 and 22 would theoretically decrease the River overflow by nearly 70 percent with an attendant 83 percent BOD_5 and 77 percent TSS reduction. However, as illustrated in Table 33 and Figure 41 eliminating overflows from areas nos. 7, 16 and 22 and increasing regulator capacities to handle peak flows reduces the total River overflow by only 15 percent. Similarly, eliminating overflows from areas nos. 7, 16, 22, 8, 9 and 25 results in approximately a 50 percent reduction in total overflow. Table 32 contains information helpful in explaining these discrepancies. When drainage area overflows are blocked, the downstream areas are surcharged even when the St. Paul Interceptor constraints are removed. The drainage areas downstream on the interceptor system therefore experience a greater overflow than when not surcharged. For example, when drainage areas nos. 7, 16 and 22 are blocked, the overflow volume increases in areas nos. 8, 9, 21, 25, and Front Street while decreasing at the Siphon. As a result the overflow reduction is only 14.9 mil gal instead of a theoretical 23.4 mil gal.

Blocking selected overflows appears to create the greatest surcharge problem for area no. 21. From Table 33 it is obvious that drainage area no. 21 is where nearly all system relief occurs when other drainage area overflows are blocked. This table helps to explain the attenuated flow in the St. Paul Interceptor.

As seen in Figure 40 the proposed tunnel interceptor system would generate a peak flow through the St. Paul Interceptor of 260 cfs for the 2 Yr-2 Hr storm. The duration of the peak lasts for approximately 40 min. Increasing river regulator capacities threefold greatly increases the peak flows from the individual drainage areas, yet the St. Paul Interceptor peak flow is not significantly increased. In fact, the total overflow reduction is only about 7 percent when the average peak flows from individual areas are more than doubled. Increasing all regulator capacities by 150 percent increases overflows at areas nos. 21 and 25, even when the St. Paul Interceptor constraints are removed. This backflow in areas nos. 21 and 25 attenuates the effect of the increased 7 mil gal of overflow to the St. Paul Interceptor. Upon increasing the regulator capacity by 150 percent and removal of the interceptor constraints, the peak flow is only 265 cfs distributed over a period of 6 hr. Moreover, increasing both the regulator and interceptor capacity results in reduced overflows and essentially no change to peak flows to the Van Lare STP.

Blocking overflows from drainage areas nos. 7, 16, 22 and at the same time increasing the respective regulator capacities to the expected peak flow at the regulator (from the 2 Yr-2 Hr storm), results in a reduction in the total River overflow of about 15 percent. The additional 15 mil gal which reaches the Van Lare STP is spread out over a period of slightly more than 7 hr at a peak rate of 270 cfs. By increasing the blocked overflow regulator capacities to the predicted peak flow, no flooding occurs at the regulators. Blocking overflows and increasing

TABLE 33. SELECTIVE BLOCKAGE OF OVERFLOWS PROPOSED TUNNEL INTERCEPTOR SYSTEM 2YR-2HR DESIGN STORM

Areas No. 7, 16, 22, 8 & 9, and 25 blocked, regulator capacity unaltered, regulator capacity @ 7, 16, 22, 8 & 9, and 25 increased to peak, weirs unaltered, St. Paul Interceptor Increased Capacity			Areas No. 7, 16, & 22 blocked, regulators unaltered, weirs unaltered, St. Paul Interceptor Unaltered			Overflows not blocked regulators unaltered, weirs unaltered, St. Paul Interceptor Improved			
Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Drainage Area No.
-	330	N	-	23	Y	9.1	20	Y	7
-	328	N	-	79	Y	8.4	76	N	16
-	292	N	-	40	Y	5.8	38	N	22
-	765	Y	42.4	91	Y	20.3	153	N	8,9
20.4	72	N	16.4	159	N	11.9	86	N	21
-	328	N	9.3	21	N	8.6	21	N	25
10.5	-	N	7.2	-	N	6.2	-	N	Front
14.9	-	N	11.0	-	N	8.5	-	N	Siphon
10.0	-	N	9.9	-	Y	9.8	-	N	28
20.4	-	N	20.9	-	Y	20.9	-	N	29
10.4	-	N	10.5	-		10.5	-	N	31

Note: Y = Yes, N = No, M = Maybe

regulator capacities to peak flows therefore appears to be more effective than just increasing regulator and interceptor capacities.

Figure 42 illustrates flow patterns through a typical overflow regulator for drainage area No. 22, for the 2 Yr-2 Hr storm. It is seen that increasing the regulator capacity 150 percent and removing the St. Paul Interceptor constraints results in a reduction in overflow of 2.3 mil gal (40 percent reduction) from this area. The remaining 60 percent is removed by blocking the overflow and increasing the regulator capacity to the expected maximum flow. The peak flow has a very short duration of only 20 min. Increasing regulator capacities and eliminating interceptor constraints decreases overflows and results in no flooding at the regulator. However, merely plugging the overflow without any improvements to the regulators or the St. Paul Interceptor results in heavy flooding in the lower portion of the drainage area. This situation is evident in Table 33. As seen, the reduction in total River overflow is the same when blocking drainage area nos. 7, 16 and 22 overflows regardless of whether the regulators or St. Paul Interceptor capacities are increased; however, when the capacities are not increased, flooding occurs. The results would therefore indicate that increasing both the capacity of the regulators and the St. Paul Interceptor are required to effectively reduce potential flooding.

Selective Overflow Weir Evaluation

Raising the elevation of overflow weirs, although not always as effective, is an alternative to completely blocking the overflows. In fact, increasing the weir depth to the depth of the interceptor is analogous to blocking the overflow. For those storms that do not create full depth flows in the trunk sewers tributary to a regulator, weir elevation increases are not as effective as blockage for surcharge conditions. This latter condition is illustrated by Figure 42 and Table 34 which is similar to those used in the discussions of selective overflow blockage for the 2 Yr-2 Hr storm.

Figure 43 shows that raising weir elevations to interceptor crowns as well as increasing regulator capacities to peak flows for all River overflows except Front St. and the River Siphon is as effective as blocking overflows from areas nos. 7, 16 and 22. The total River overflow for increasing the weirs is 83.4 mil gal as opposed to 86.3 for blocking overflows. Raising all River overflow weirs, Siphon excluded, and increasing their regulator capacities 150 percent results in an equivalent overflow (83.6 mil gal); however, this condition causes flooding at certain regulators (See Table 32). Raising weir elevations only at drainage areas nos. 7, 16, 22 and 25 (increasing regulator capacities to peak flows) results in a total River overflow of 95.9 mil gal or about one third the overflow reduction experienced when blocking the overflows in areas nos. 7, 16, and 33. Thus, this is not as effective as increasing all river regulator capacities 150 percent which results in a total River overflow of 94.6 mil gal. The feasibility of raising weir elevations appears to be limited in light of the above.

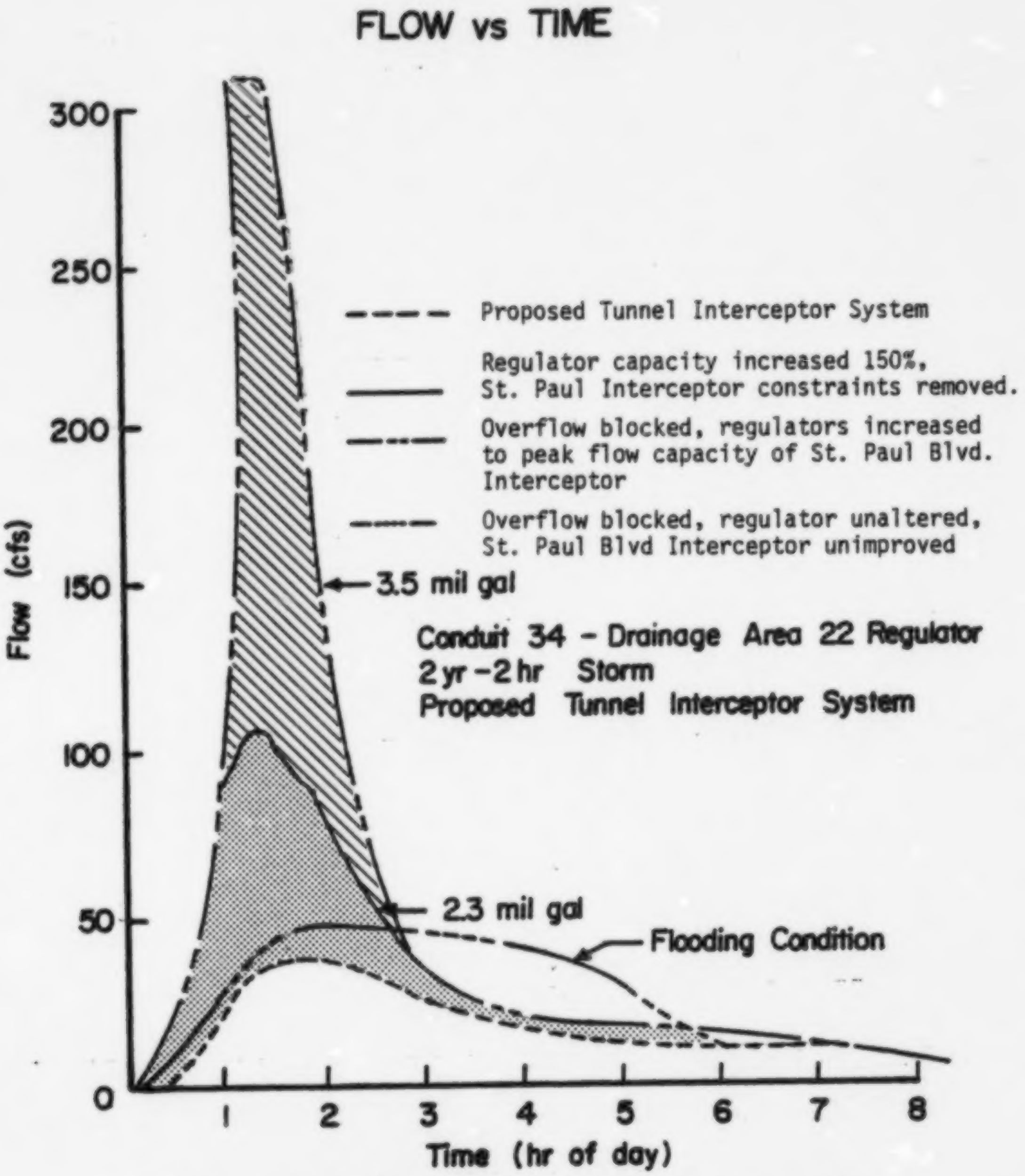
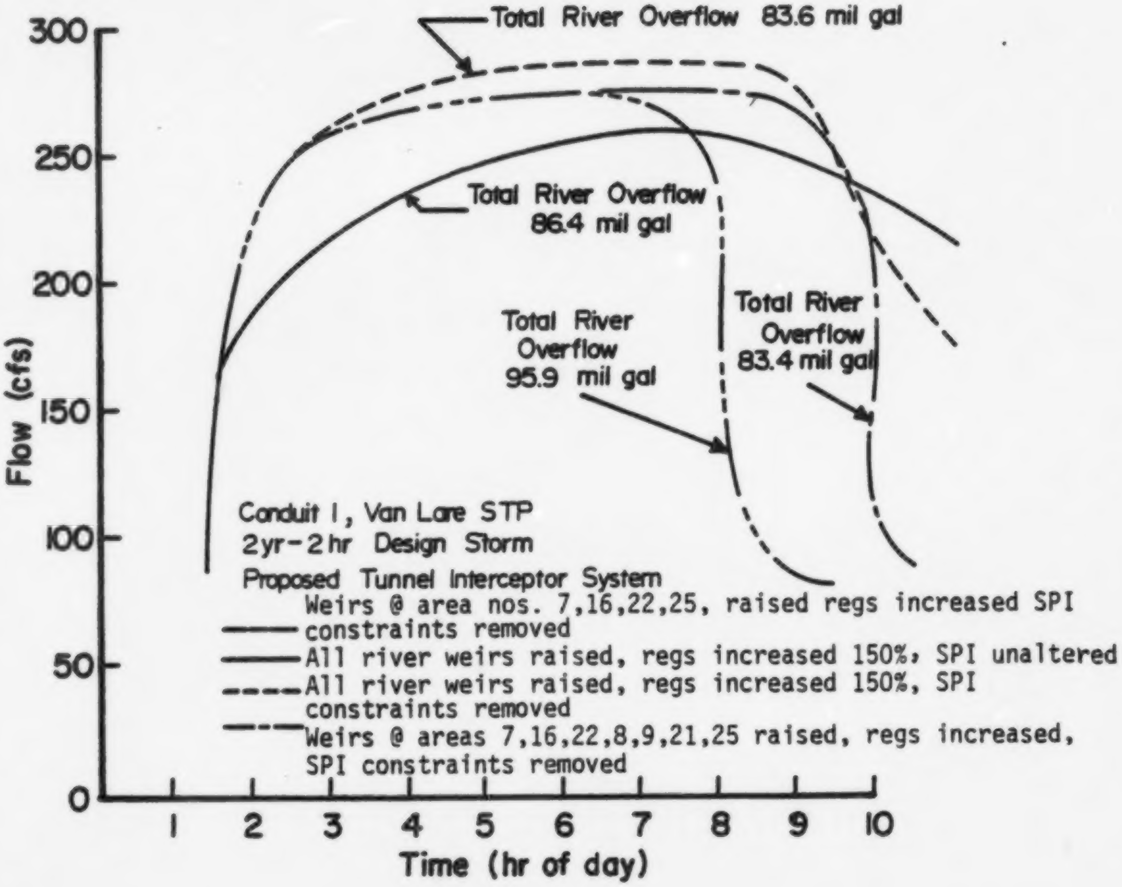


FIGURE 42. Minimal Structural Alternative Modeling Results Overflow Hydrograph Area No. 22 Regulator.

TABLE 34. SELECTIVE WEIR ANALYSIS PROPOSED TUNNEL INTERCEPTOR SYSTEM 2 YR-2 HR STORM

		Regulator capacities increased 150% weirs all increased to depth of interceptor St. Paul Interceptor constraints unaltered			Regulator capacities increased 150% weirs all increased to depth of interceptor St. Paul Interceptor constraints removed			Weirs @ drainage areas 7, 16, 22 and 25 raised regulators @ these drainage areas increased to peak flow St. Paul Interceptor Constraints removed			Weirs & Drainage areas 7,16,22, 25,8&9 and 21 increased to depth of interceptor regulators & these drainage areas increased to peak flow St. Paul Interceptor constraints removed		
Drainage Area No.	Overflow Conduit No.	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator	Overflow/ Volume (mil gal)	Peak Flow @ Regulator (cfs)	Flooding @ Regulator
7	6	6.4	65	Y	6.4	65	Y	1.3	319	N	1.3	323	N
16	25	4.2	223	N	4.1	224	N	1.4	391	N	1.4	391	N
22	35	3.7	110	N	3.5	111	N	0.3	308	N	0.6	283	N
8&9	31	34.7	208	M	33.7	232	M	40.5	125	M	30.8	935	M
21	214	4.1	244	N	9.2	235	N	16.8	83	N	11.8	217	N
25	40	10.7	61	N	9.8	61	N	7.9	323	N	8.2	323	N
Front St.	250	7.8	-	N	6.3	-	N	20.9	-	N	21.7	-	N
Siphon	251	15.0	-	N	10.7	-	N	6.7	-	N	7.6	-	N

Note: Y = Yes, N = No, M = Maybe



Note: Regs = Regulators, SPI = St. Paul Blvd. Interceptor

FIGURE 43. Minimal Structural Alternative Modeling Results
Flow Projections at the Van Lare STP

Raising overflow weir elevations requires increasing regulator capacities in order to significantly reduce the total system overflow to the Genesee River. Under similar regulator and interceptor conditions, overflow blocking results in a better reduction of the 2 Yr storm River overflow than raising weir elevations, and therefore, would decrease River overflows at least as well as raising weirs for any storm less than the 2 Yr storm. In summary, raising weir elevation, as opposed to overflow blockage, does not appear to be a feasible alternative to reducing combined sewer overflows.

Regulator Capacity Evaluation

Increasing regulator capacity has been discussed under minimal structural alternatives. It has been shown that increasing regulator capacities is more effective than raising weirs and that increasing regulator capacities helps alleviate flooding when overflows are blocked. In this subsection the effect of just increasing regulator capacities will be discussed. Figure 44 shows the relationship between total River overflow and percent increase in regulator capacity using the proposed tunnel interceptor system with the St. Paul Interceptor constraints removed. Table 35 shows the overflow from individual River regulators which is associated with the annual analysis.

TABLE 35. TOTAL RIVER OVERFLOW VS. REGULATOR CAPACITY BASED ON SIMPLIFIED STORMWATER MODEL

	REGULATOR CAPACITY INCREASE				
Drainage Area No.	0%	50%	100%	150%	200%
7	22.4	5.0	0.8	0.0	0.0
8 & 9	392.1	116.8	37.4	3.0	2.7
16	0.0	0.0	1.0	14.8	14.8
21	0.0	0.0	0.0	0.0	0.0
22	9.5	0.0	0.0	4.9	5.2
25	3.9	1.3	6.0	14.1	14.1
Front St.	0.2	1.3	2.9	3.1	3.1
Totals (mil gal)	428.1	124.4	48.1	39.9	39.9

Previous work has shown that increasing regulator capacities on the present interceptor system results in an asymptotic relationship whereby increasing regulator capacities by more than threefold results in no appreciable change in overflow reduction. Again, this phenomenon is apparent on the proposed tunnel interceptor system. The analysis using the 2 Yr storm shows only minimal overflow response to regulator increases, but the yearly analysis dramatically expresses the response in overflow to the River to changes in regulator capacities. Beyond a 150 percent regulator capacity increase, no appreciable overflow reduction occurs. Figure 44 shows that annual total River overflow can be reduced by about 95 percent by increasing regulator capacities 150 percent and removing the St. Paul Interceptor constraints.

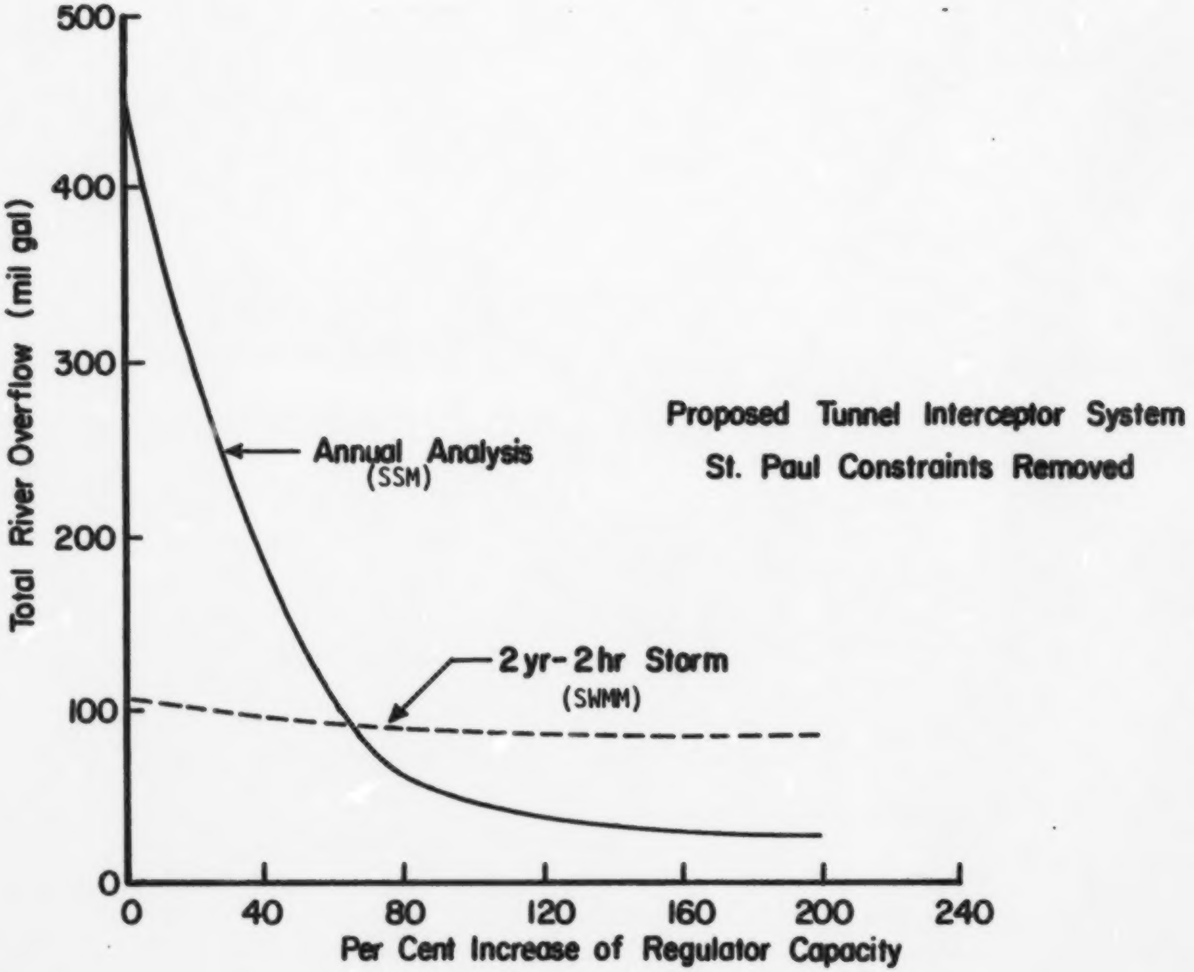


FIGURE 44. Minimal Structural Alternative Modeling Results Total River Overflow vs Regulator Capacity

Table 35 presents the response of individual drainage areas to regulator capacity increases and reveals an interesting point. As capacities increase the total River overflow decreases, but the River overflows upstream of drainage areas nos. 8 and 9 (specifically nos. 16, 22, 25 and Front St.) increase. This is due to the influence of large flows leaving the regulator from drainage areas nos. 8 and 9 on the upstream hydraulics.

STRUCTURALLY INTENSIVE ALTERNATIVES

Alternate 1

This alternate considers capturing the first-flush from all the River overflow sites and treating all post first-flush flows with primary swirl devices. The temporarily stored first-flush volume would be released back into the existing interceptor system on termination of system overflows. Treatment for this first-flush volume would be at wet-weather treatment facilities located at the Van Lare STP. Also entering this plant for wet-weather treatment would be flows from the St. Paul Interceptor and the Cross-Irondequoit Tunnel. This alternate is shown schematically in Figure 45.

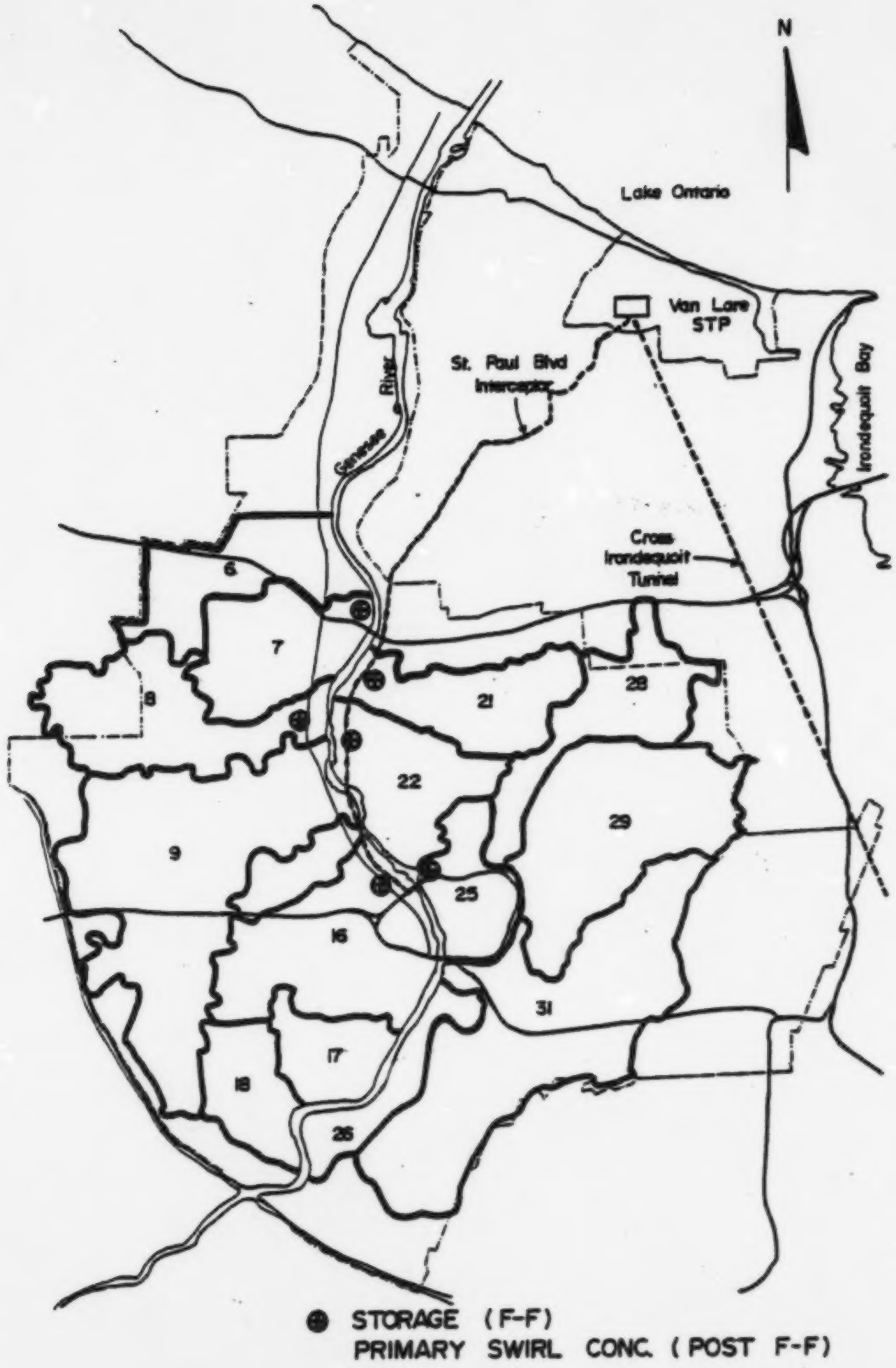
The cost/benefit analysis for this alternate is presented in the following section. First-flush capture and post first-flush swirl performance quantities are presented in Section 7.

Alternate 2

This alternate involves evaluating various storage and treatment options for capturing the total overflow to the Genesee River. The treatment plant under consideration would be located on the Genesee River in the vicinity of the lower falls. The influent to the plant would be the St. Paul Boulevard overflows conveyed by individual conduits from each drainage area regulator. By controlling the rate of treatment, storage within these conduits would be affected. This alternate is shown schematically in Figure 46.

Two different approaches to analyzing this alternate were made utilizing the SWMM. The 2 Yr-2 Hr design storm provided the required rainfall input to the system. Computer simulation with the SWMM involved 3600 integration steps of 10 sec each. It is believed that such a short time interval provides an accurate representation of the flowrates within the sewer system. Table 36 shows the runoff and overflow volumes associated with each drainage area under the application of the design storm.

The first approach involved the balance of storage and treatment assuming the St. Paul Interceptor has continuous flow to the Van Lare STP throughout the rainfall-runoff event. The initial analysis also did not incorporate improvements to the interceptor system. The low regulator capacity from each drainage area to the St. Paul Interceptor was increased by 150 percent. Under this initial analysis, the overflows resulting from the east side of the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Lake Ontario
Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
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8
21
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22
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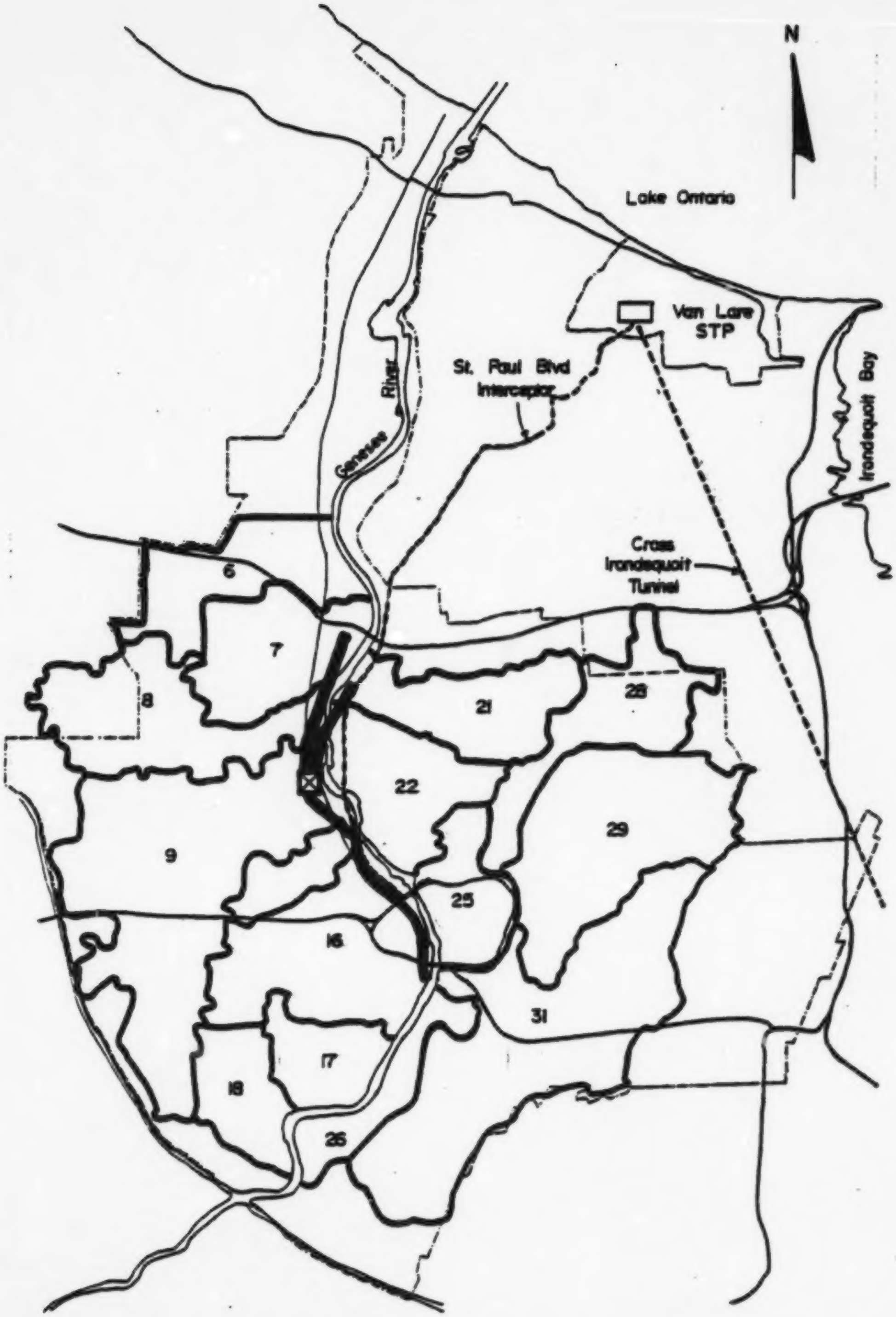
9
25
16
31
17
18
26
⊕ STORAGE (F-F)
PRIMARY SWIRL CONC. (POST F-F)

FIGURE 45. Schematic of Structurally Intensive Alternate No. 1 - Storage First-Flush/ Treatment Post First-Flush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Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
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OVERFLOW INTERCEPTOR
⊠ OVERFLOW TREATMENT PLANT

FIGURE 46. Schematic of Structurally Intensive Alternate No. 2 - Overflow Treatment Along Genesee River

TABLE 36. RUNOFF AND OVERFLOW VOLUMES BY DRAINAGE AREA AS PROJECTED BY THE SWMM FOR THE 2 YR - 2 HR DESIGN STORM

Drainage Area No.	Runoff (mil gal)	Overflow (mil gal)
7	11.25	9.00
8 & 9	54.22	41.41
16	14.11	8.51
21	8.42	8.43
22	8.99	5.80
25	8.78	8.90
Front St. (from nos. 17, 18, 26 and part of no. 31)	28.77	6.46
Siphon	-	12.95
Totals	134.54	101.55

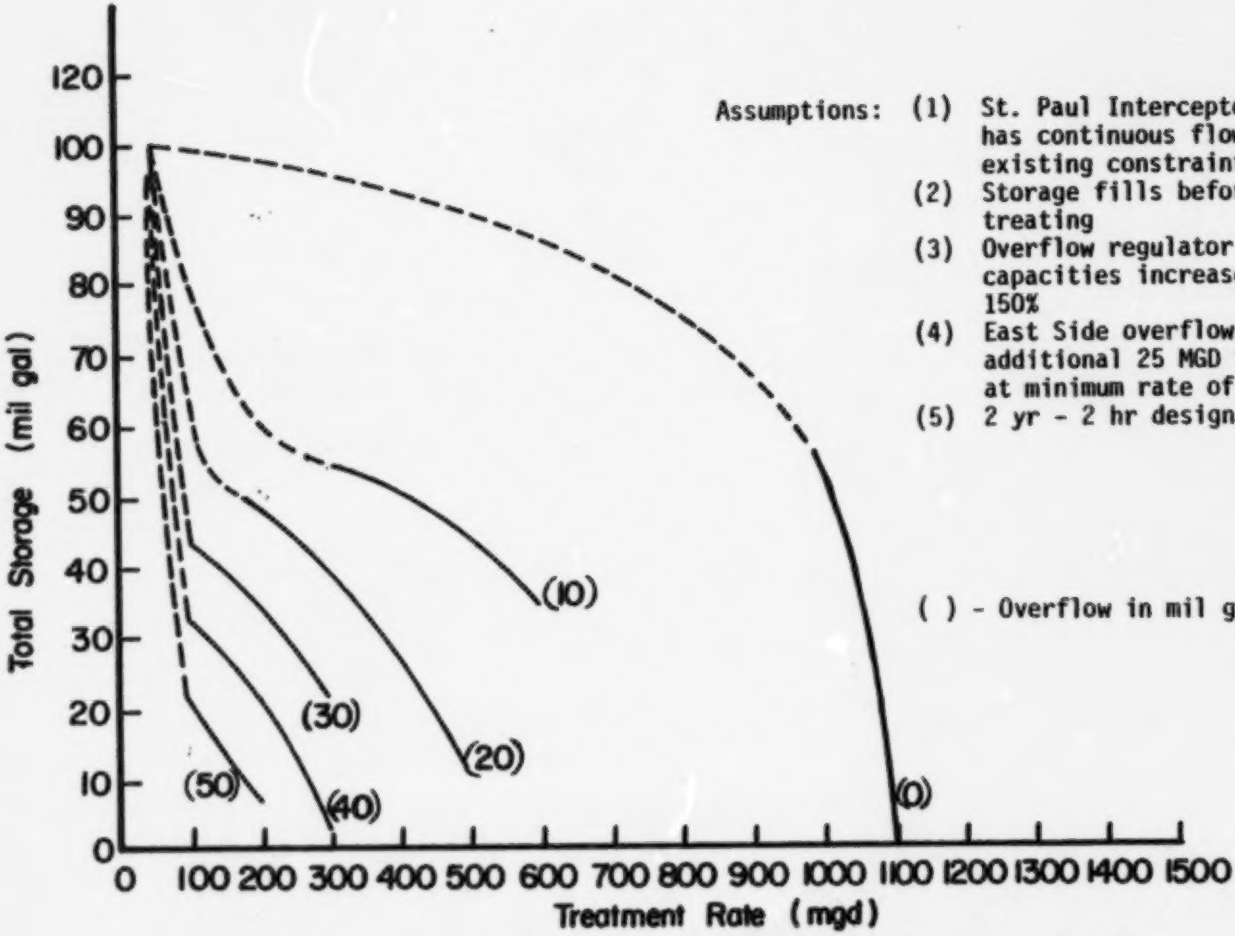
Total Rainfall 2 yr - 2 hr storm = 1.15 in.
Total Rainfall over the drainage areas tributary to River = 286.40 mil gal
Gross Runoff Coefficient = 0.47

city that discharge to the Cross-Irondequoit Tunnel were stored and treated at a maximum rate of 45 mgd at the Van Lare STP. The last condition which separates this approach from the second, is that the river overflow interceptor conveying all overflow volume to the treatment facility is allowed to fill completely before pumping and treatment begin. Figure 47 shows the various combinations of storage capacity and treatment rates necessary to limit the various overflow volumes. It should be noted that the overflow volumes represent wastewater discharges to the various receiving waters contributed by system relief and treatment plant by-pass induced by insufficient storage or treatment capacities.

Figure 47 shows that as the treatment rate increases, the required storage capacity decreases for a given overflow volume. The necessary storage requirements for a given treatment rate can be translated into a specific diameter tunnel by using Figure 48. This figure shows the necessary diameter tunnel for any given storage capacity based on a tunnel length of 18,800 ft. This represents the approximate length of the individual conduits leading to a River treatment facility from the various drainage area regulator overflow points.

From a quality consideration the total pollutant load reaching the treatment facility from the river overflow conduits for the design storm is 70,930 lb and 173,450 lb of BOD_5 and TSS, respectively. Total overflow volume is 101.55 mil gal.

The second approach taken to analyze this alternative was to balance storage and treatment under the same conditions as the first approach with the exception that flows from the individual river overflow conduits would



Assumptions: (1) St. Paul Interceptor has continuous flow with existing constraints
(2) Storage fills before treating
(3) Overflow regulator capacities increased by 150%
(4) East Side overflows and additional 25 MGD treated at minimum rate of 45 MGD
(5) 2 yr - 2 hr design storm

( ) - Overflow in mil gal

FIGURE 47. Storage vs Treatment Requirements for Varying Overflow Volumes for Centrally Located River STP

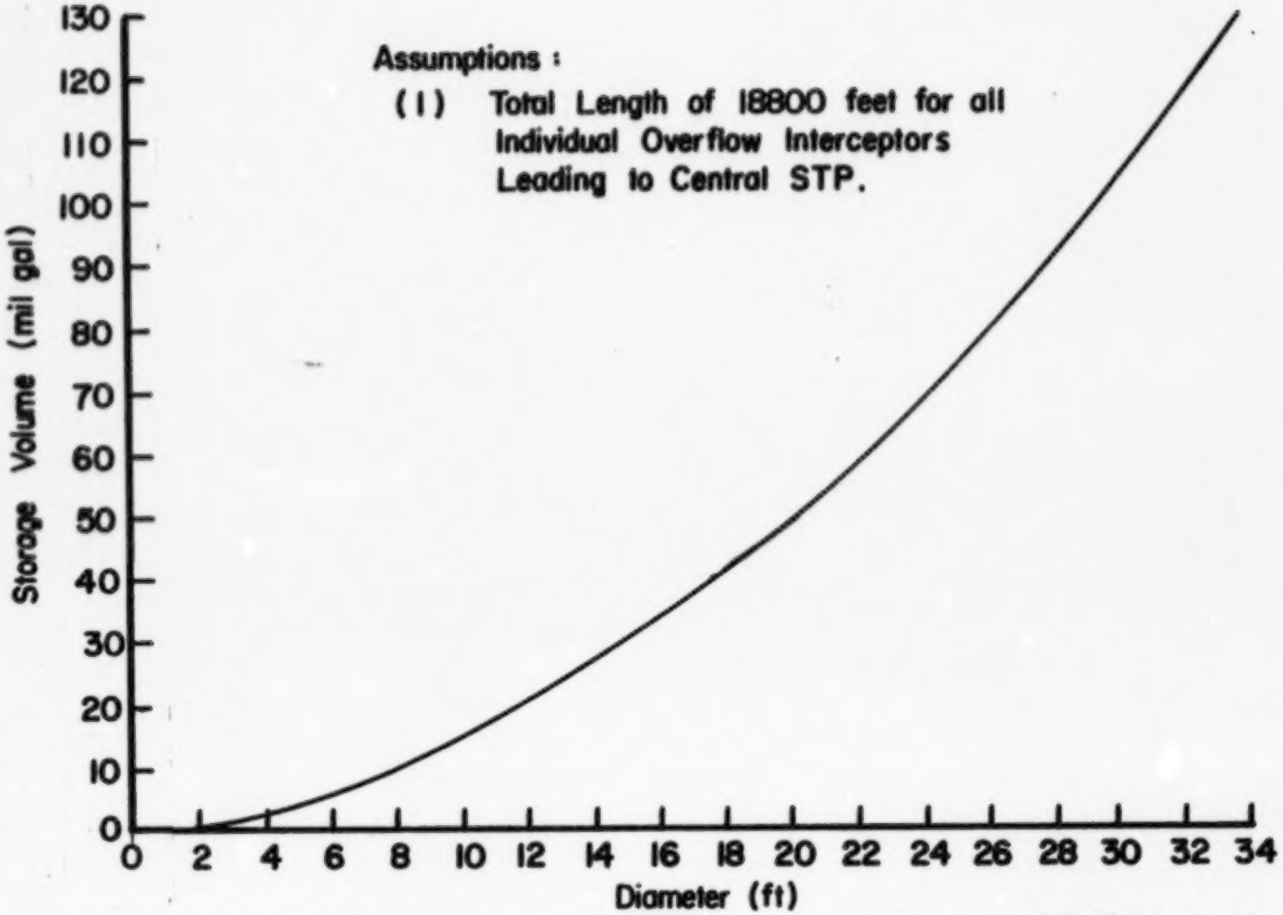


FIGURE 48. Storage Volume vs Tunnel Diameter for Structurally Intensive Alternate No. 2

be pumped and treated as they reach the treatment facility. That is, treatment would begin on initiation of the overflow event. The second approach to the analysis of Alternate 2 is based on the mass diagram of the inflow to the river treatment facility. This method is similar to the Rippl procedure (19) for the determination of storage required in impounding reservoirs. Figure 49 presents the cumulative mass inflow curve for the river treatment facility. Superimposed on the mass curve are straight lines representing various treatment rates. The maximum vertical distance between the mass curve and a particular treatment rate line is the maximum amount of storage capacity necessary to prevent an overflow or a plant bypass. For varying overflow quantities, Figure 50 shows the relationship between storage and treatment for several river overflow conduits conveying overflows to a wet-weather facility located on the river.

Given the amount of storage capacity required for a specific treatment rate, Figure 48 shows the necessary tunnel diameter for this storage volume. As previously indicated this figure is based on an assumed total length of river overflow conduits of 18,800 ft.

Comparing Figures 47 and 50 it is seen that less storage capacity is required for a given treatment rate for the second approach than for the first. Since treatment begins with the collection of the first overflow volumes instead of proceeding after the filling of the interceptor, it is obvious that the second approach is a more cost-effective method for this alternative.

The cost/benefit analysis for this alternate is shown in the following section. Storage and treatment relationships for the analysis are based on curves shown on Figure 50.

Alternate 3

An analysis similar to that presented for Alternate 2 was made on the proposed tunnel sewer system incorporating Alternate 3. The difference between Alternate 2 and 3 is the location of the treatment facility. Under Alternate 3, the treatment facilities are located at the present Van Lare STP. This alternate is shown schematically in Figure 51. As with Alternate 2, two approaches were taken for analysis. The first approach involved the River overflow interceptor leading to the treatment plant filling completely before pumping and treatment. Figure 52 shows the relationship between storage capacity and treatment rate for various overflow volumes. The second approach involved treating the overflows as they occurred. Figure 50 of Alternate 2 shows the relationship between storage and treatment for this condition. These two figures indicate that the second approach is a more cost-effective method. For a given treatment rate less storage capacity is required when treating from the start of the overflow event than when allowing the overflow interceptor to fill before treating. In both cases, Figure 53 shows the size of storage and conveyance tunnel necessary for a given amount of storage. This figure is based on an interceptor length of 36,000 ft which represents the approximate length of the river overflow interceptor leading to the Van Lare STP along a route adjacent to the St. Paul Interceptor originating from the Front Street overflow site.

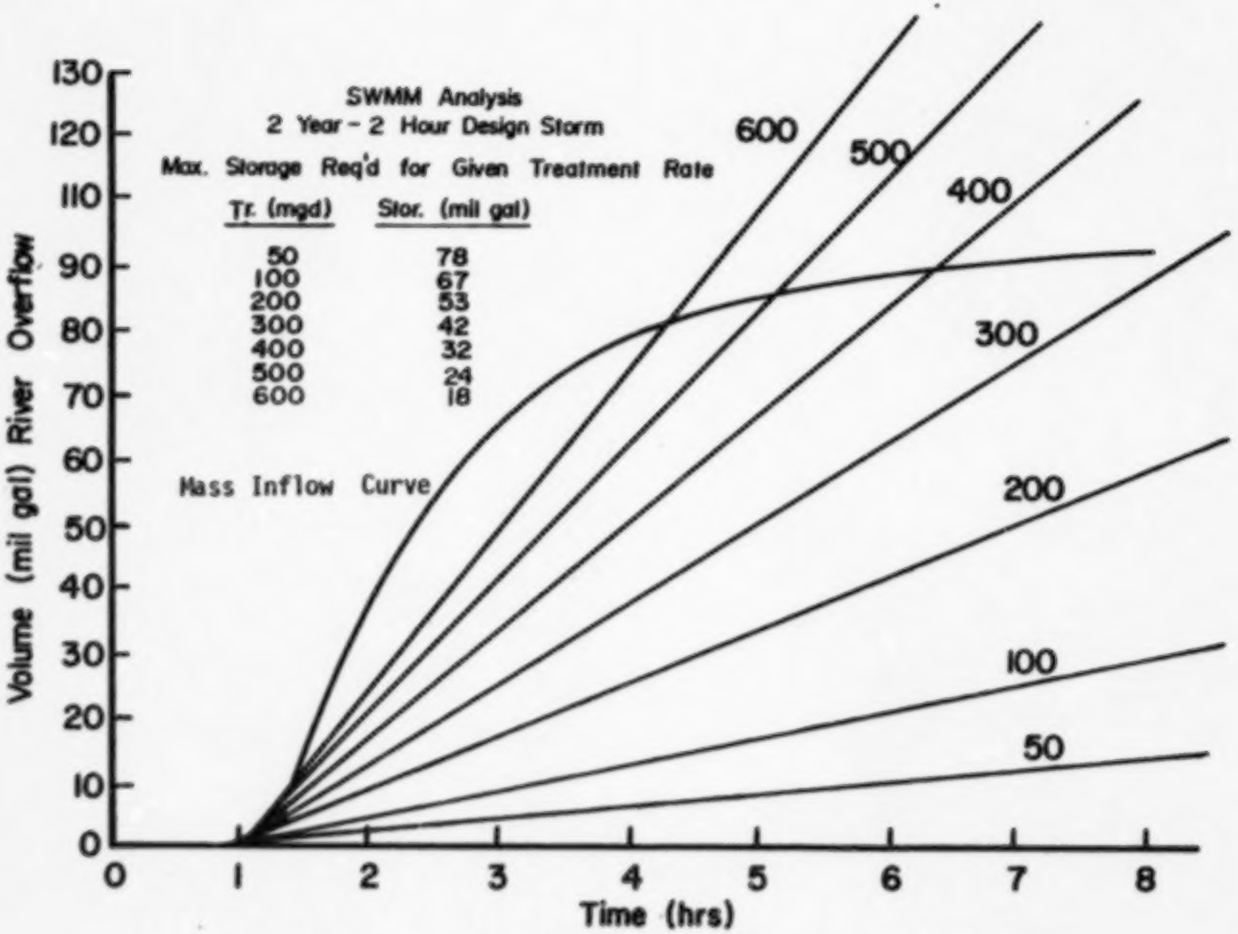


FIGURE 49. Mass Inflow Curve for the River Treatment Facility

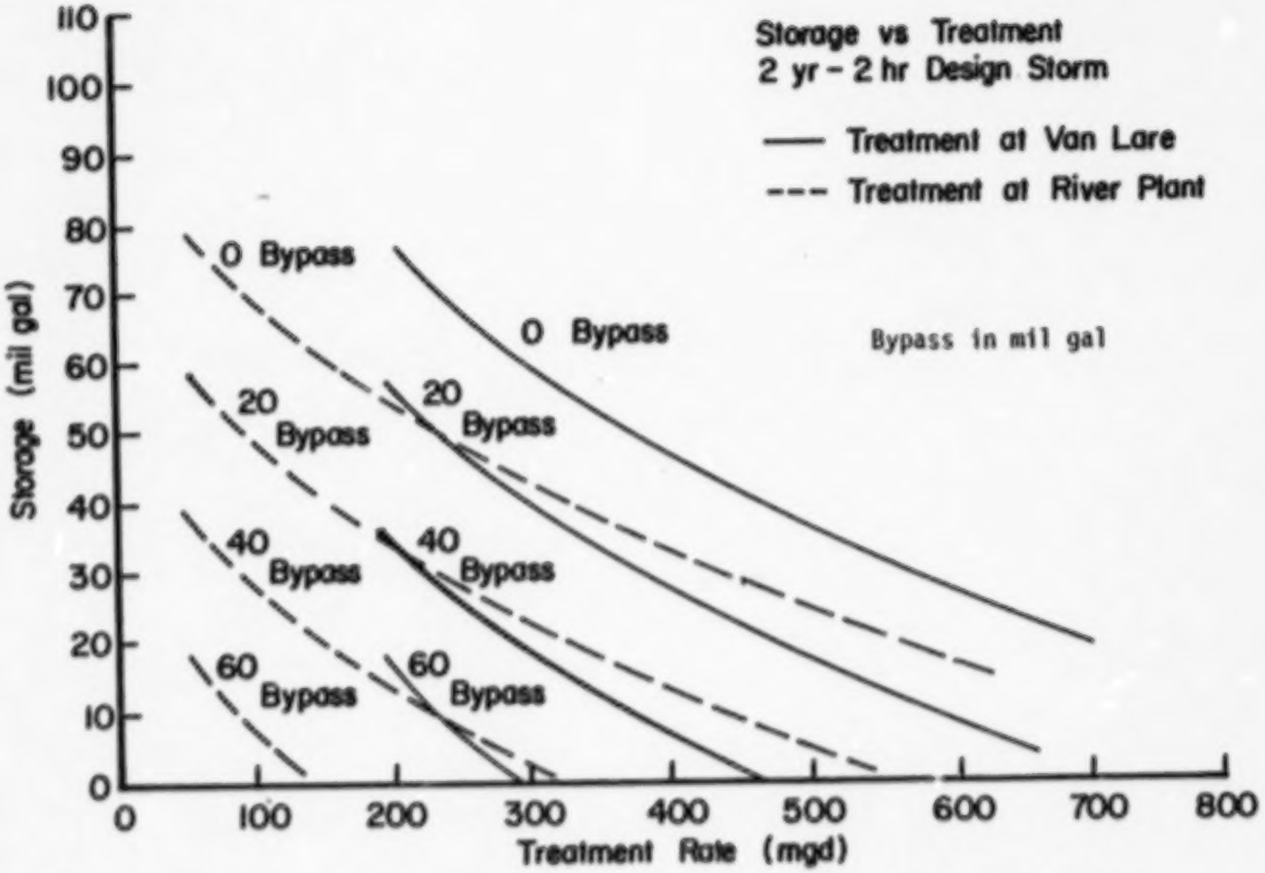
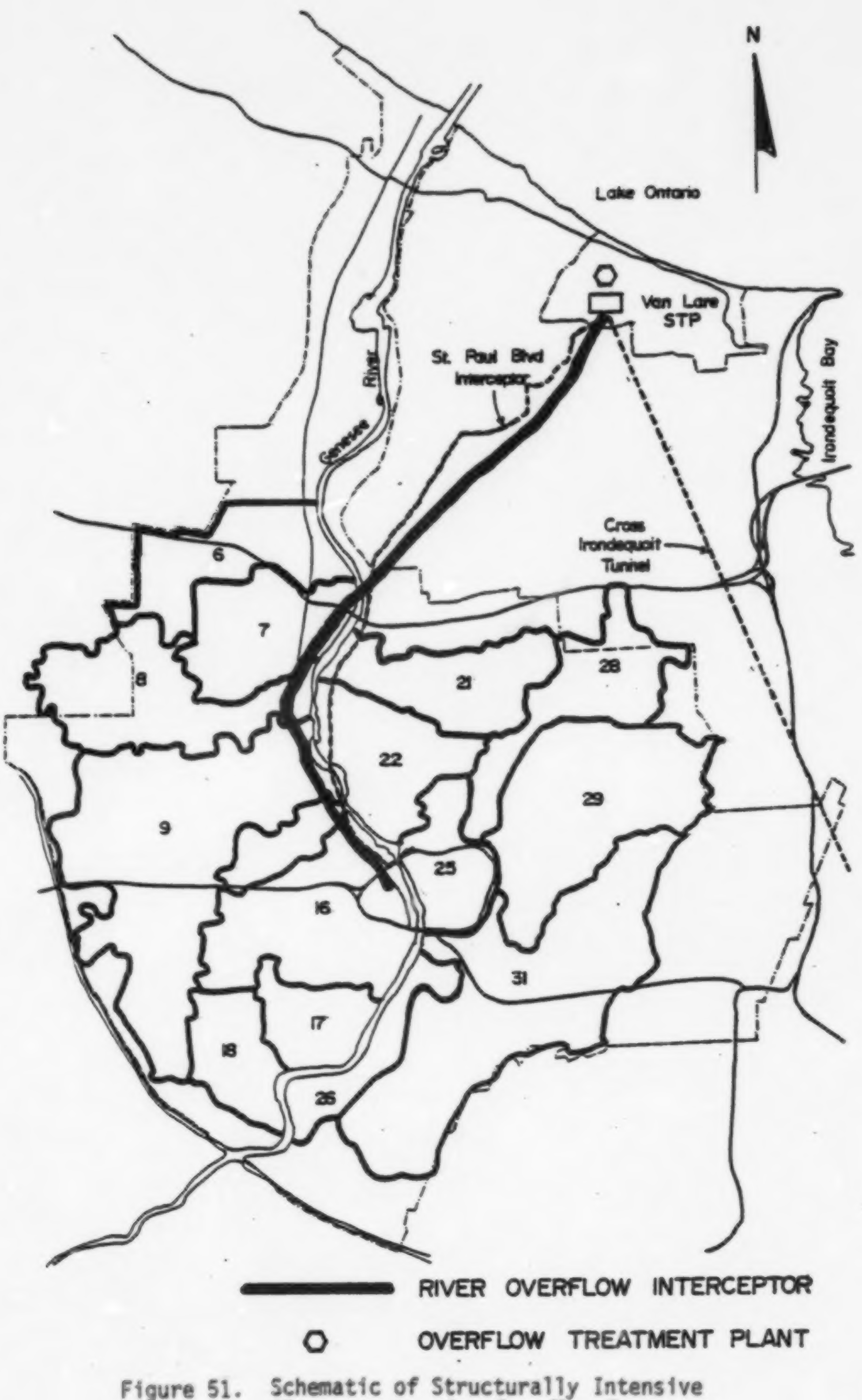


FIGURE 50. Storage vs Treatment Rate for Varying Overflow Quantities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St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
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RIVER OVERFLOW INTERCEPTOR
OVERFLOW TREATMENT PLANT

Figure 51. Schematic of Structurally Intensive Alternate No. 3 - Overflow Treatment at Van Lare STP.

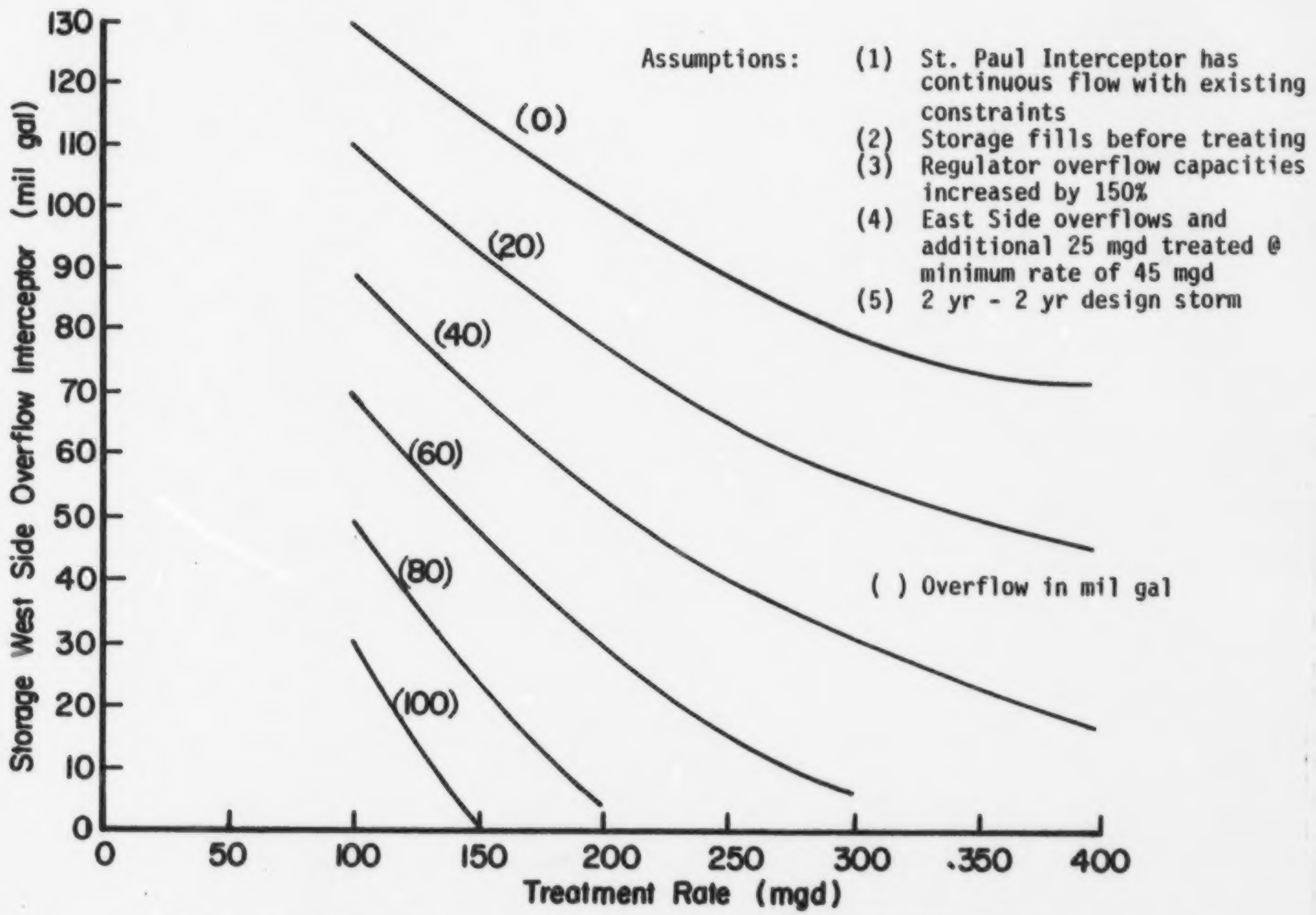
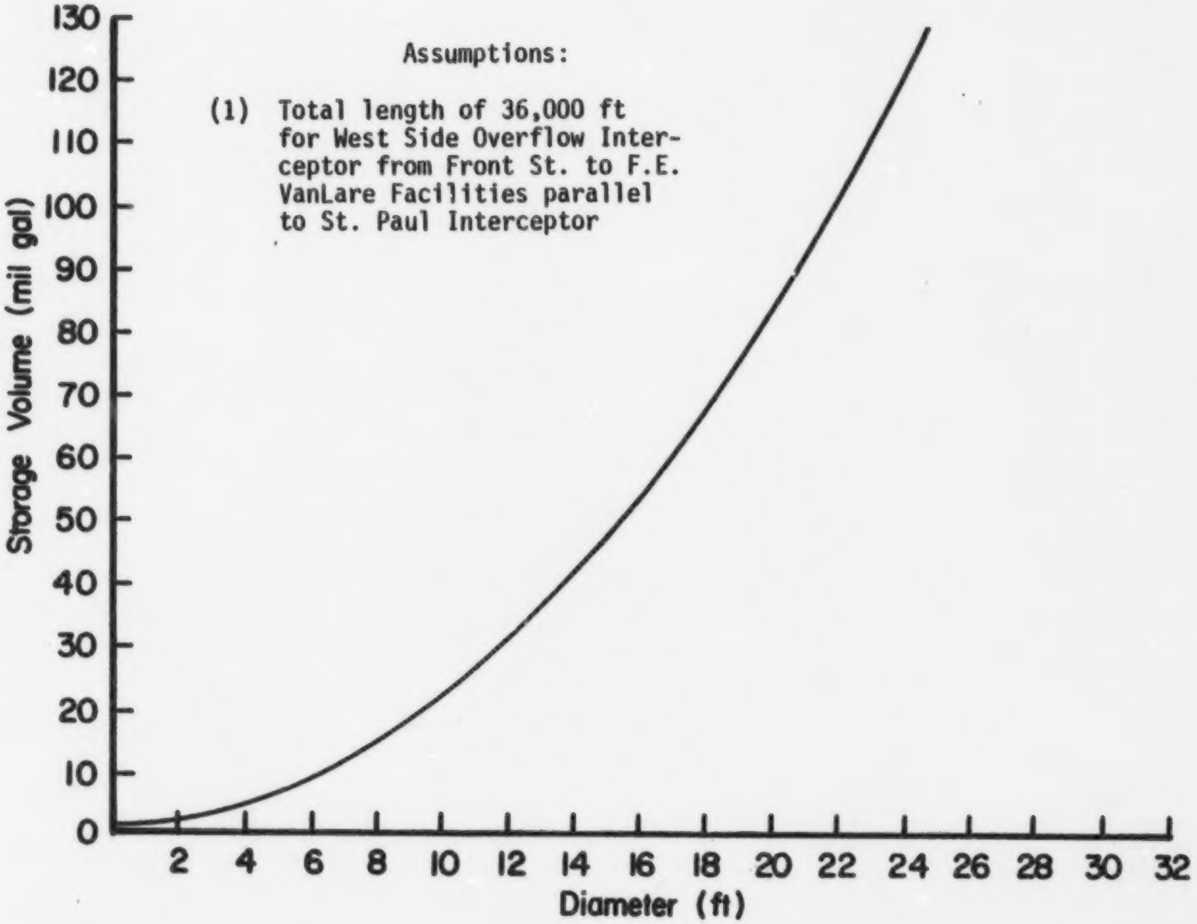


FIGURE 52. Storage vs Treatment for Varying Overflow Volumes for Treatment Plant Located at F.E. Van Lare



Assumptions:

(1) Total length of 36,000 ft for West Side Overflow Interceptor from Front St. to F.E. VanLare Facilities parallel to St. Paul Interceptor

FIGURE 53. Storage Volume vs Tunnel Diameter for Structurally Intensive Alternate No. 3

The cost-effective analysis for this alternate is presented in Section 9. Storage and treatment relationships for the cost-effective analysis are based on Figure 50.

Alternate 4

This alternate is similar to Alternate 1 with the exception that the post first-flush is not treated but is directly discharged to the River. This alternate is shown schematically on Figure 54.

The cost-benefit analysis is presented in the following section. First flush capture and post first-flush discharge quantities are based on calculations developed in Section 7.

Alternate 5

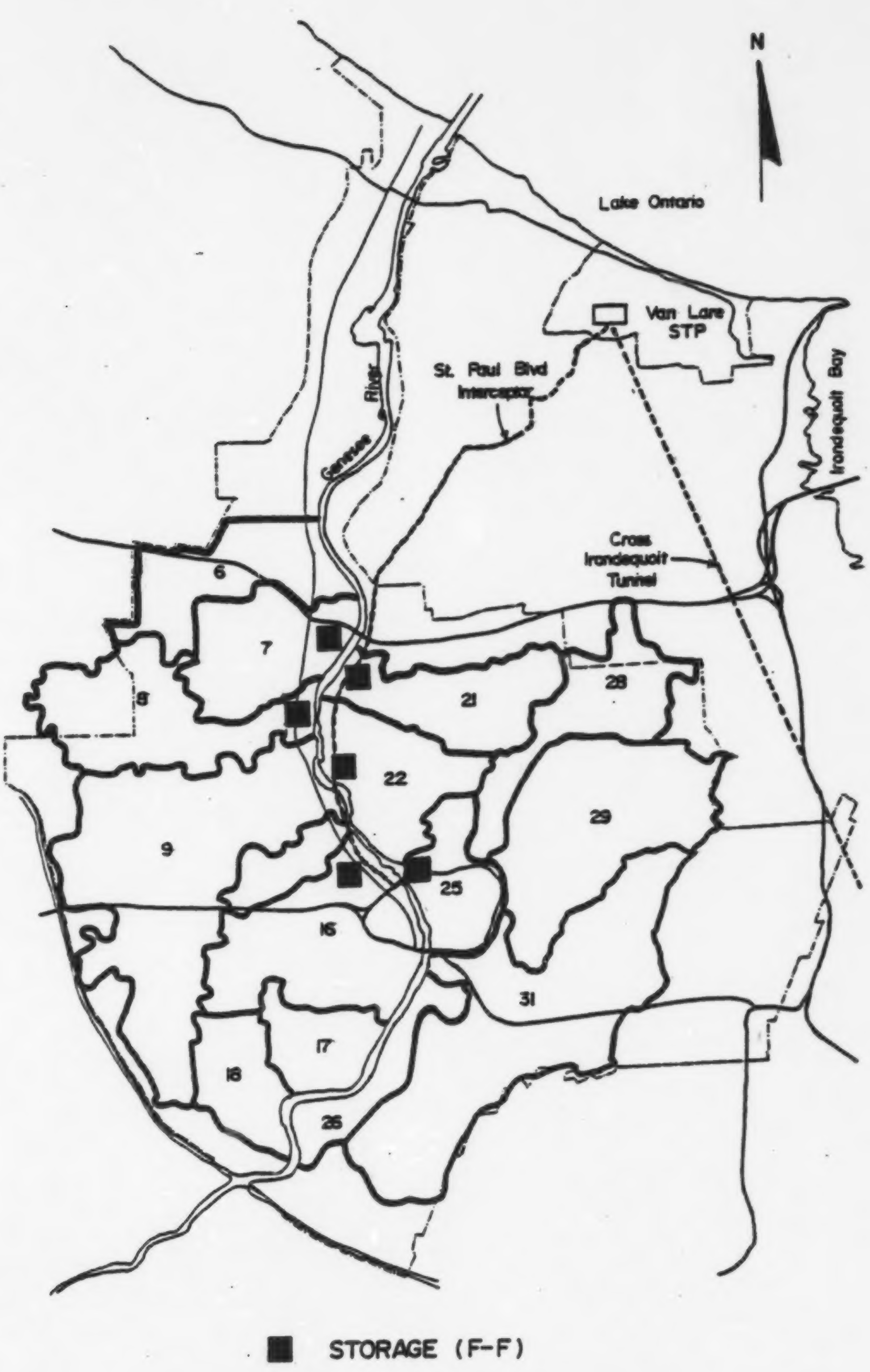
This alternate considers the use of primary swirl concentrators on each of the River overflow locations for treatment of the entire overflow volume. The swirl effluents are discharged directly to the River. The Van Lare STP would treat flows entering from the St. Paul Interceptor and the Cross-Irondequoit Tunnel. This alternate is shown schematically in Figure 55.

Based on calculations presented in Section 7, providing a swirl unit at each of the overflow sites reduces total river suspended solids loading by 33 percent - from 173,450 to 116,220 lb under influence of the design storm. Table 37 shows performance of the swirl units in reducing the pollutant load to the River.

TABLE 37. PRIMARY SWIRL SOLIDS REDUCTION TREATING ENTIRE OVERFLOW 2 YR - 2 HR DESIGN STORM

Drainage Overflow No.	Primary Swirl Influent Load lb	Load to River lb	Reduction %
7	14540	8590	41
8,9	58900	39660	33
16	23750	10410	56
21	5220	5170	1
22	24000	21590	10
25	12770	11010	14
Front	4790	4790	0
Siphon	29480	15000	49
	173450	116220	33

The size of each swirl concentrator was based on the analysis presented in Alternate 1. These sizes are shown in Table 25 of Section 7. Since a swirl unit at Front Street yielded no reduction in suspended solids, further analysis of this alternate involved primary swirl units of the specified diameter at all overflow locations except Front Street.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Van Lare STP
Irondequoit Bay
St. Paul Blvd Interceptor
Genesee River
Cross Irondequoit Tunnel
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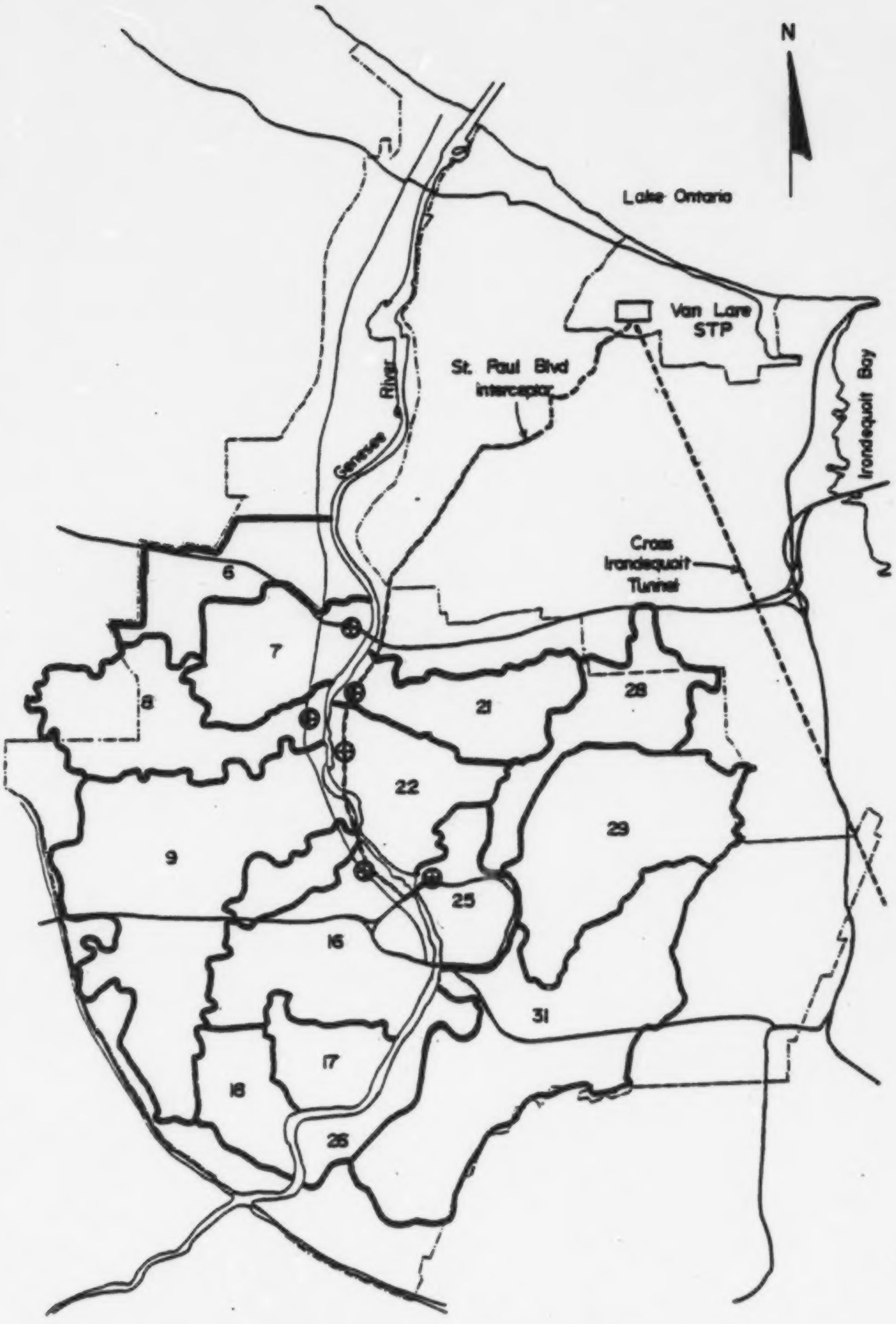
■ STORAGE (F-F)

FIGURE 54. Schematic of Structurally Intensive Alternate No. 4 - Storage of First-Flush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Van Lare STP
St. Paul Blvd interceptor
Genesee River
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Cross Irondequoit Tunnel
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⊕ PRIMARY SWIRL CONCENTRATOR
(TOTAL FLOW)

FIGURE 55. Schematic of Structurally Intensive Alternate No. 5 - Local Overflow Treatment

The cost/benefit analysis of Alternate 5 is presented in Section 9.

Alternate 6

This alternate involves the evaluation of a storage and treatment balance for intercepting the river overflows and conveying them to the Cross-Irondequoit Tunnel for treatment at the Van Lare STP. This alternate attempts to take advantage of the storage capacity available on the east side of the system to attenuate west side flows by providing a connecting interceptor tunnel. The feasibility of this alternative will be shown using the SSM for determining the relationship between storage treatment, and overflow. This alternative is shown schematically in Figure 56.

The model analysis was made on a drainage area of 12,073 acres with a gross runoff coefficient of 0.45. Available storage amounting to 84.8 mil gal representing the Cross-Irondequoit and the Culver-Goodman Tunnels was evaluated using varying treatment rates to predict overflow frequency and volumes. Table 38 shows the storage-treatment-overflow relationship for this alternative on an annual basis.

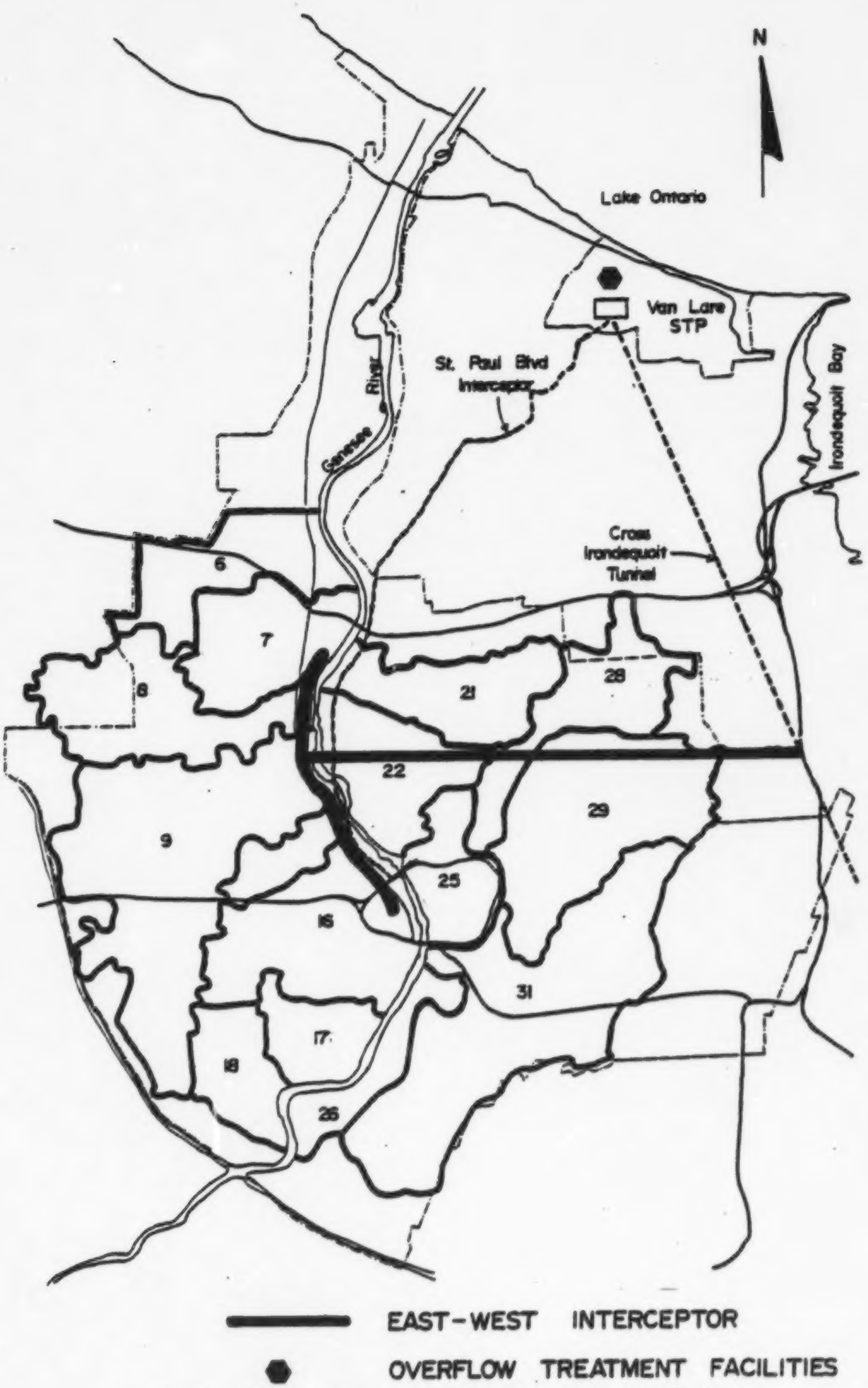
TABLE 38. WEST-EAST STORAGE-TREATMENT BALANCE SIMPLIFIED STORMWATER MODEL

Total Catchment Area 12073 Acres
Gross Runoff Coefficient 0.45
Storage 84.8 mil gal

Treatment Rate (mgd)	Average Annual Overflow	
	Days	Volume (mil gal)
10	35.30	1494.90
50	6.00	315.76
100	1.90	104.34
150	0.70	35.90
200	0.30	8.90

Note: Storage of 84.8 mil gal represents available storage in the Cross-Irondequoit and Culver-Goodman Tunnel Complex.

The cost/benefit analysis is presented in Section 9.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Van Lare STP
St. Paul Blvd Interceptor
Genesee River
Irondequoit Bay
Cross Irondequoit Tunnel
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EAST-WEST INTERCEPTOR
OVERFLOW TREATMENT FACILITIES

FIGURE 56. Schematic of Structurally Intensive Alternate No. 6 - Conveyance of Overflow by East-West Interceptor

SECTION 9

COST-EFFECTIVE ANALYSIS OF STRUCTURALLY INTENSIVE ALTERNATIVES

ALTERNATIVES EVALUATED

This section presents the relative ranking of a number of alternatives in terms of capital cost versus performance. Cost/performance is represented based on costs developed for all storage, transmission, and treatment facilities and performance is represented as the total impact of all facilities on the receiving waters for the design 2 Yr-2 Hr storm.

Two categories of treatment trains were considered: (a) primary treatment and (b) two-level treatment employing filtration. Treatment trains evaluated are as shown on Table 39. All treatment trains included disinfection.

TABLE 39. PROCESS TRAIN CONFIGURATIONS

A. Primary Treatment

1. *G/S + DIS
2. #P/S + DIS
3. **F/S @ 800 gpd/ft^2 + DIS
4. F/S @ 1500 gpd/ft^2 + DIS
5. F/S @ 2000 gpd/ft^2 + DIS

B. Two-Level Treatment

1. G/S +¢DMF + DIS
2. P/S + DMF + DIS
3. F/S @ 800 gpd/ft^2 + DMF + DIS
4. F/S @ 1500 gpd/ft^2 + DMF + DIS
5. F/S @ 2000 gpd/ft^2 + DMF + DIS

* swirl degritter

# swirl primary separator

** flocculation/sedimentation

¢ dual media filtration

The alternative area-wide configurations were presented in Section 8. Analyses of Alternates 1, 4 and 5 were for single cases each involving the following assumptions:

Alt. 1 West side first-flush overflows are stored in 34.88 mil gal facilities and routed to 148 mgd treatment facilities (2000 gpd/ft^2 F/S with alum and polymer) at the Van Lare STP which also handles east side overflows. West side post-first-flush overflows are treated by primary swirl concentrators designed to handle peak flow.

Alt. 4. Same as Alt. 1, except west side post-first-flush overflows are discharged untreated.

Alt. 5. Same as Alt. 1, but all west side overflows to be treated by the local swirl devices. East side flows to be sent to the Van Lare STP.

Alternate 2 (treatment of west side overflows at a central plant on the Genesee River) and Alternate 3 (treatment of all area overflows at a central plant at the Van Lare STP) involved evaluation of many subalternatives. The cost/benefit analysis included evaluation of several process trains (one-and two-level treatments), storage vs treatment optimizations, and allowance for various amounts of untreated overflow bypass. Computer modeling of costs and performance allowed evaluation of 576 subalternatives within these two alternatives.

DEVELOPMENT OF COST RELATIONSHIPS

Capital cost versus treatment capacity relationships were compiled from several literature sources (8) and adjusted to a common basis to allow comparison of the various sources. From these sources relationships were selected incorporating the effect of design loadings. These curves are based on November 1976 projected costs.

The total capital cost requirements for each alternative include the costs of the assumed treatment facilities, storage, pumping, and sludge treatment. The relationships used in the cost development are shown on Table 40.

COST OPTIMIZATION

A tradeoff exists between costs of storage and treatment facilities; i.e., larger storage facilities permit smaller capacity treatment plants. Generally, an optimum combination of sizing treatment and storage facilities will permit the development of least cost facilities. In the evaluation of Alternates 2 and 3, each subalternative was evaluated for four different combinations of storage-treatment sizes. The determination of storage versus treatment requirements was discussed in Section 8. The overflow mass diagram (Figure 49) was used to compile the combinations of storage vs treatment in Figure 50 for different bypass volumes.

When storage and treatment costs were determined from the combinations derived from Figure 50, relationships similar to Figure 57 were obtained. In general, the trends of these curves showed that the more capital intensive systems (e.g., F/S at 800 gpd per ft^2 and the two-level systems) showed optimization which maximized the amount of storage and minimized the size of treatment facilities. The less capital intensive systems such as swirl concentrators showed minimum cost with minimum storage. Other systems showed minimum cost at some compromise between the maximized extremes of storage and treatment capacities.

TABLE 40. PROCESS COST MODELS

A. Primary Treatment:

1. G/S
2. P/S — Cost functions presented in detail in Volume II
3. F/S

B. Pumping from storage to Interceptor
$\$ \text{ mil} = 0.185\ Q_D^{0.484}$

C. Disinfection System
$\$ \text{ mil} = 0.011\ Q_D^{0.851}$

D. Sludge Pumping Cost
$\$ \text{ mil} = 0.0274\ Q_D^{0.5}$

E. Sludge Treatment
$\$ \text{ mil} = 4.73$ (assumed independent of Q_D - based on average storm conditions for volumes of CSO treated)

F. Filtration System
$\$ \text{ mil} = 0.23\ Q_D^{0.753}$

G. Tunnel Storage
Tunnel
$\$/\text{ft} = 35.80\ (\text{dia in ft})^{1.252}$
Drop Shaft
$\$ \text{ mil} = 2.23$ each

H. Force Main Cost

1. Van Lare STP Alt. $\$ \text{ mil} = 0.625\ Q_D^{0.569}$
2. River Alt. $\$ \text{ mil} = 0.337\ Q_D^{0.569}$

Note: Q_D = design flowrate in mgd

COST-EFFECTIVE RELATIONSHIPS

The weighted performance for each system was developed by analyzing the time-variable removal of TSS associated with the application of the 2 Yr-2 Hr design storm. The projected TSS removal for each system was developed for each half-hour time step and an overall weighted TSS removal was calculated for the design storm. The performance models developed in the pilot plant study were used for this purpose (8). The weighted performances assuming constant flow and variable influent TSS were as follows:

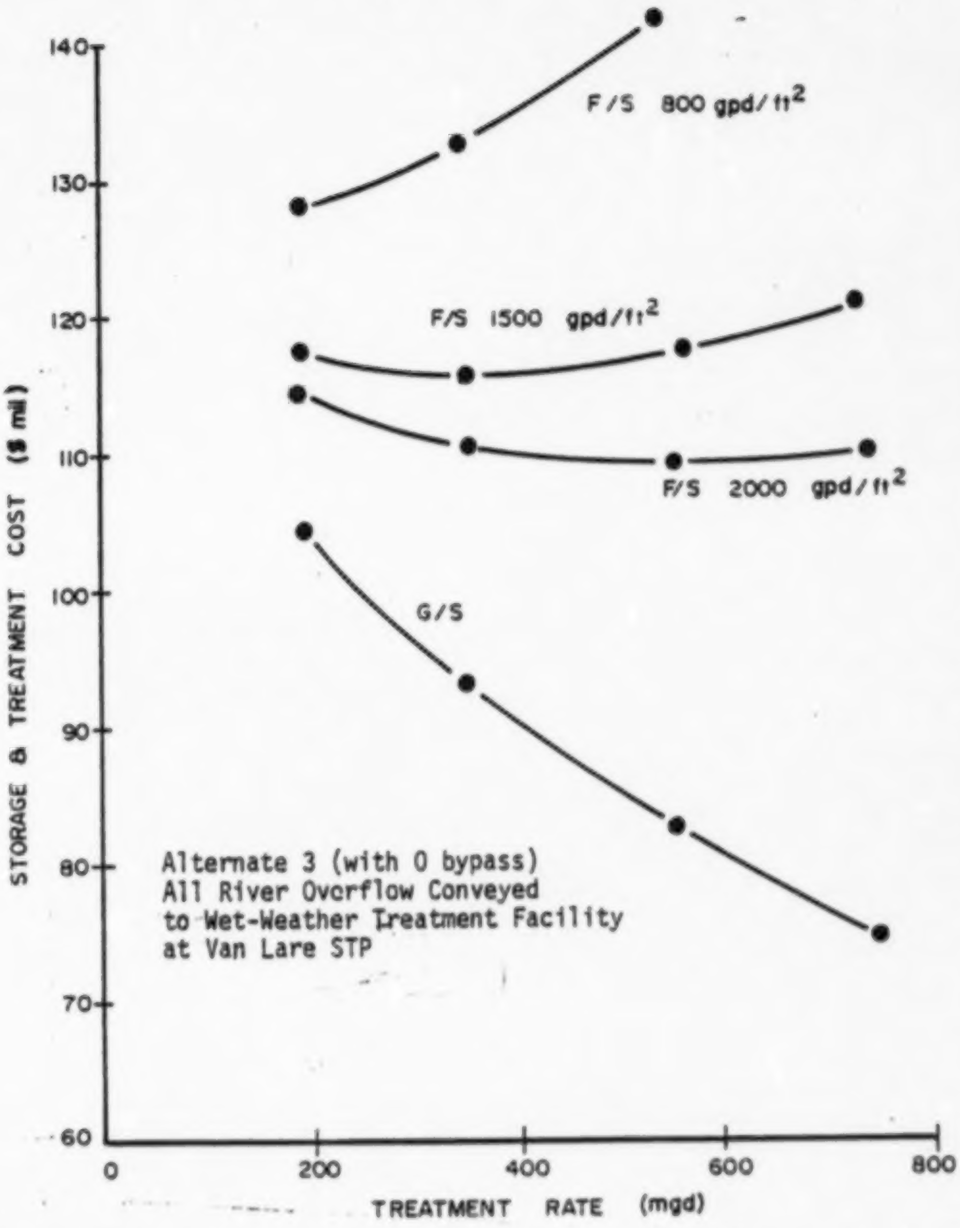

FIGURE 57. Typical Storage-Treatment Cost Optimization

	% TSS Removal
1. F/S @ 800 gpd/ft^2 with chem. trtmt.:	70.4
2. F/S @1500 gpd/ft^2 with chem. trtmt.:	66.1
3. F/S @2000 gpd/ft^2 with chem. trtmt.:	63.1
4. P/S ($D_2/D_1 = 12$, $N_f = 0.167$)	39.2
5. G/S ($D_2/D_1 = 6$, $N_f = 0.167$)	24.7
6. System 1 with filtration @ 10 gpm/ft^2	94.1
7. System 2 with filtration @ 10 gpm/ft^2	93.2
8. System 3 with filtration @ 10 gpm/ft^2	92.6
9. System 4 with filtration @ 10 gpm/ft^2	87.4
10. System 5 with filtration @ 10 gpm/ft^2	84.9

Alternates 1 and 5 (local swirl concentrators with no storage) assumed TSS removals as a function of both variable flow and influent TSS, again using the pilot plant performance model.

The cost-effective relationships for the subalternatives of Alternate 2 are shown on Figure 58. Performance is represented by the total area wet-weather TSS discharge (pounds per design storm). The total area wet-weather TSS discharge is the sum of the amounts of solids discharged to the Genesee River (in both bypass and treated effluents) and the solids in the effluent of wet-weather treatment facilities located at the Van Lare STP. The TSS discharge figures take into account that sludges from local treatment facilities are routed back to the interceptor and must be treated again at wet-weather facilities at the Van Lare STP.

The cost-effective relationships for Alternate 3 are shown on Figure 59. Figures 58 and 59 indicate that system performance becomes limited principally by type of treatment employed and amount of bypass allowed. As the discharge requirement becomes more stringent, the number of feasible available alternatives becomes limited with the subsequent increase in capital cost.

Cost-effective comparisons of all the alternatives are summarized on Figure 60. Minimum cost lines are traced for Alternates 2 and 3 for conditions of zero-bypass and variable permitted bypass. The minimum cost lines permitting bypass are about the same for Alternates 2 and 3.

Central treatment on the Genesee River (Alt. 2) is slightly more cost-effective than Alternate 3 for the zero-bypass condition. However, Alternate 2 results in the discharge of primary treated wastewater to the dissolved oxygen assimilation capacity limited Genesee River whereas Alternate 3 results in the discharge of treated wastewater to the Rochester Embayment of Lake Ontario.

The local treatment alternatives (1, 4 and 5) appear to offer no cost performance advantages over the central treatment alternatives (2 and 3).

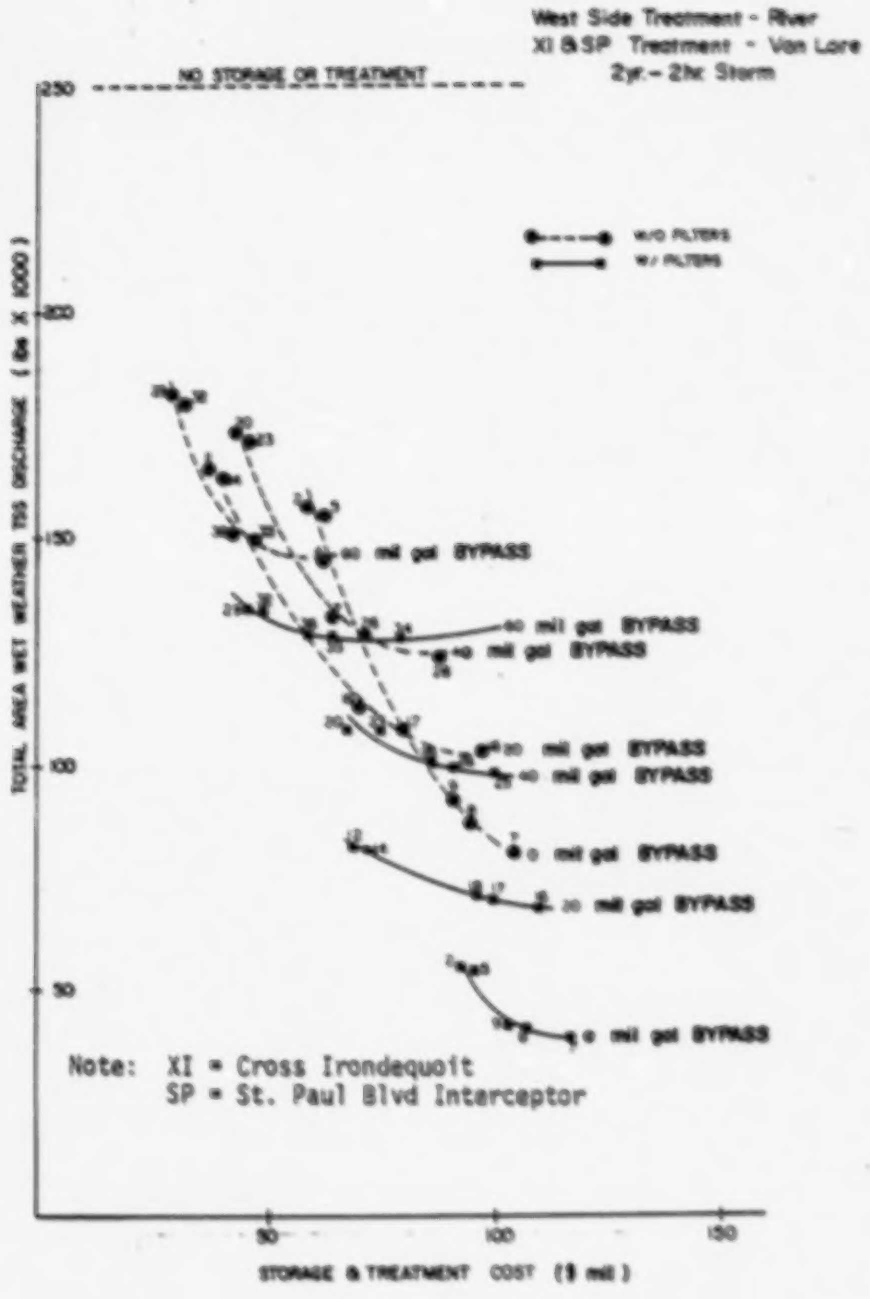

FIGURE 58. Cost-Effectiveness Relationships for Subalternatives of Alternate No. 2 - River Overflow Interceptors with Overflow Treatment Facility Centrally Located

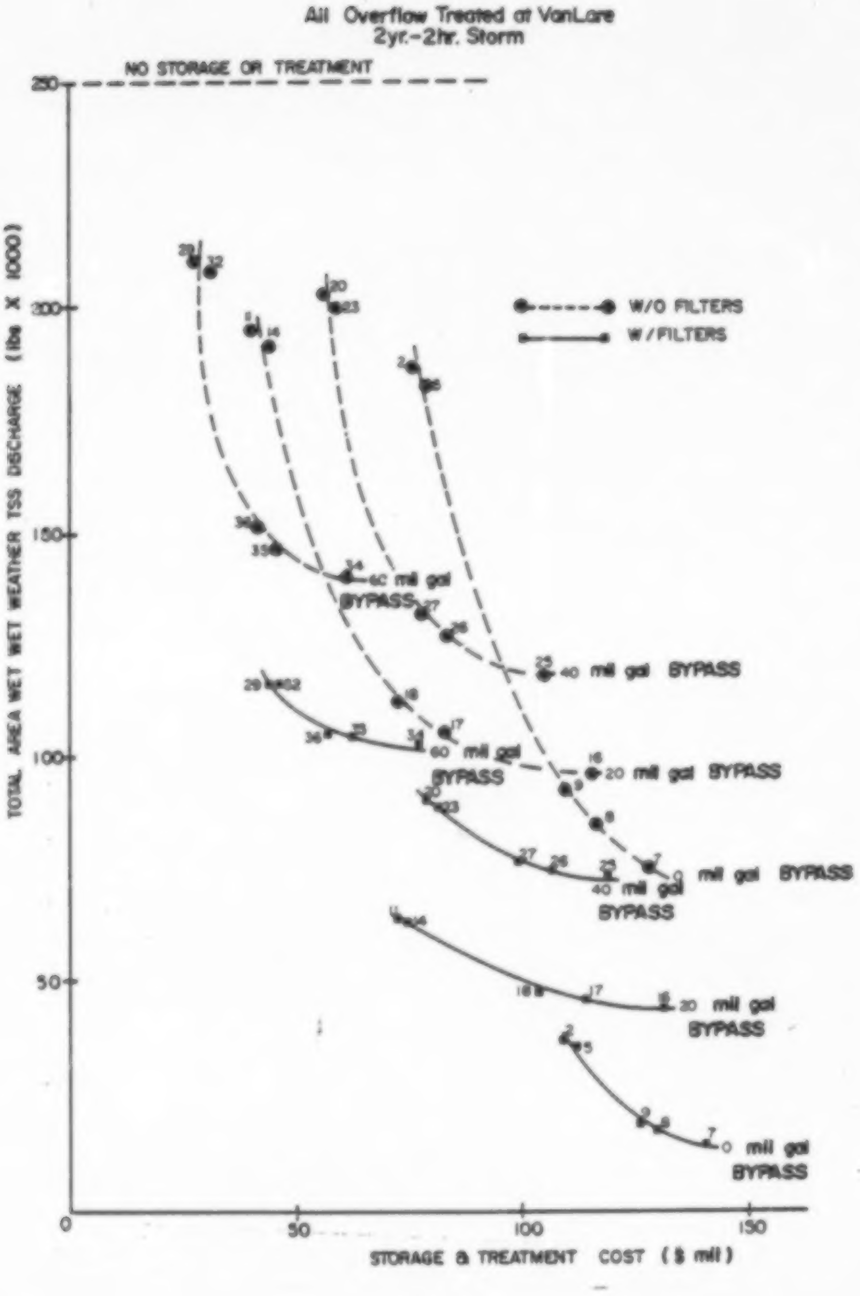

FIGURE 59. Cost-Effectiveness Relationships for Subalternatives of Alternate No. 3 - River Overflow Interceptor with Overflow Treatment Facility at Van Lare STP

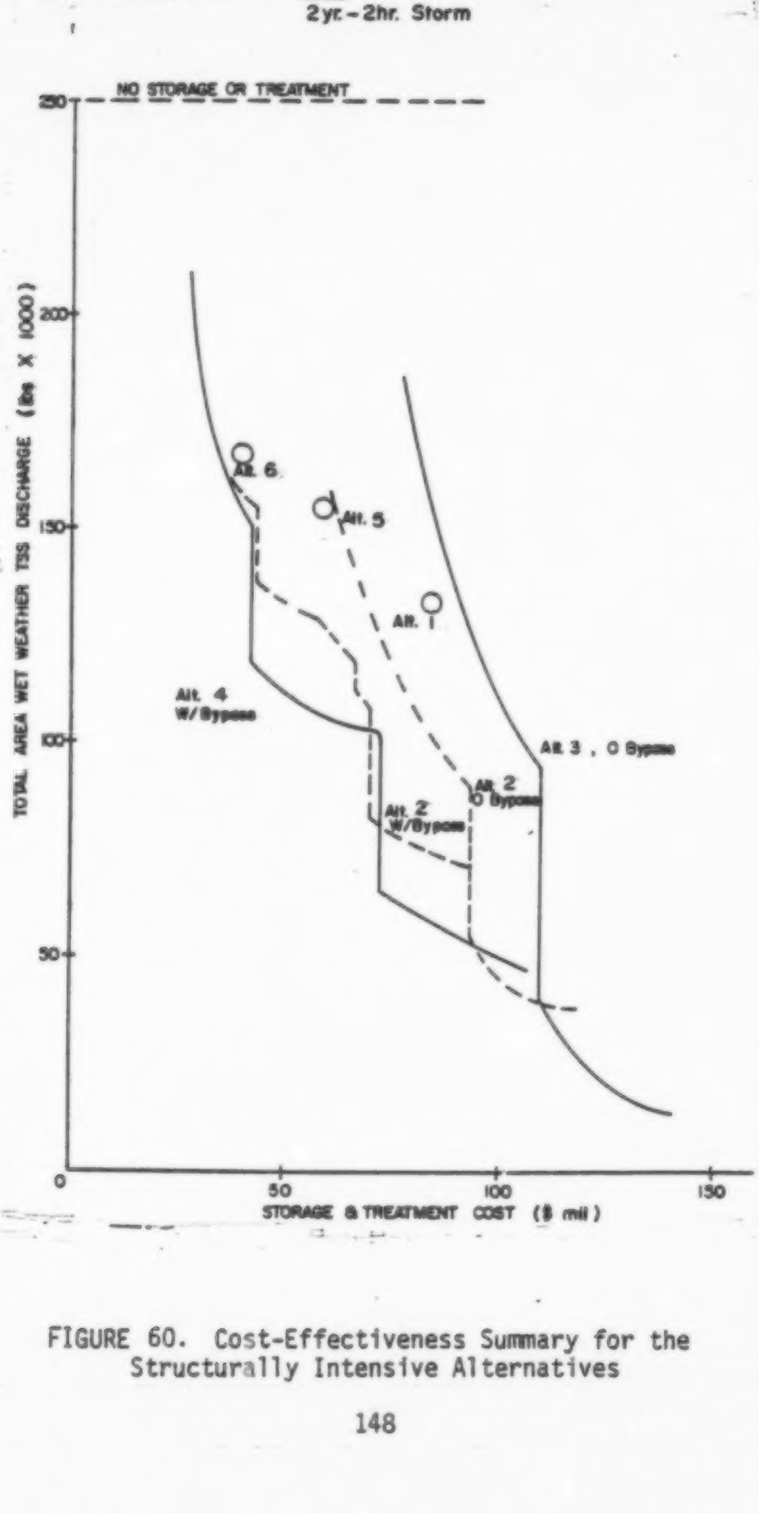

FIGURE 60. Cost-Effectiveness Summary for the Structurally Intensive Alternatives

EFFECTIVE ANALYSIS RELATIVE TO THE GENESEE RIVER

Study area loadings of TSS and oxygen demanding constituents are most critical to the water quality of the Genesee River. Water quality modeling under MA 7CD10 conditions indicates that the Genesee River can accept only very small quantities of wet-weather overflows and still maintain the dissolved oxygen standard of 4 mg/l.

The only alternative capable of completely eliminating wet-weather loadings to the Genesee River under design storm conditions is Alternate 3 for the condition of zero bypass. Figure 61 shows the cost-benefit relationship of the alternatives relative to the water quality of the Genesee River. Effectiveness is indicated as the percent reduction of solids loadings to the Genesee River. Additional loadings to the Rochester Embayment are not included in this analysis. The best reduction attainable with Alternate 2, assuming zero bypass, is approximately 93 percent. The local treatment alternatives (1, 4 and 5) result in only 32-65 percent reductions in TSS loadings to the Genesee River under application of the 2 Yr-2 Hr design storm.

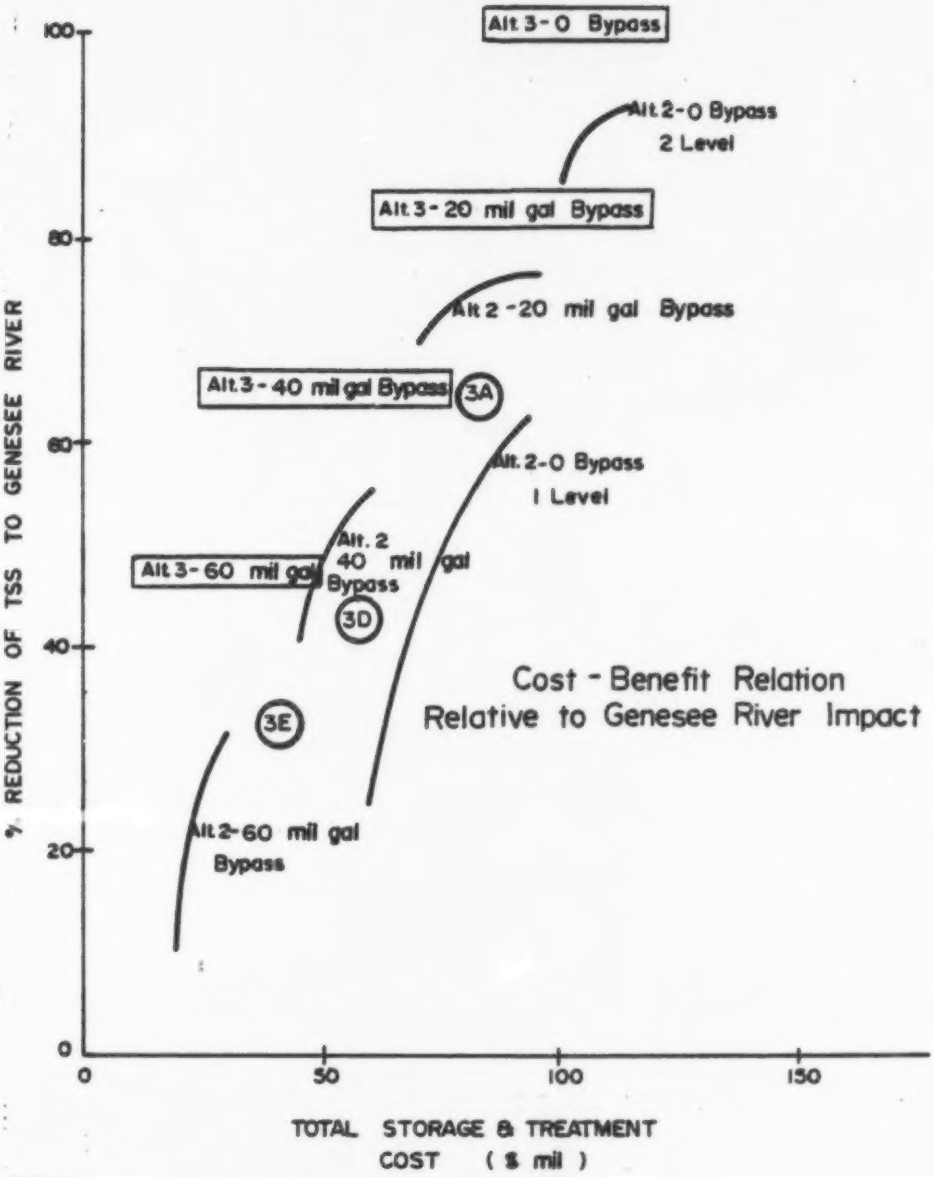

FIGURE 61. Cost-Benefit Summary for the Structurally Intensive Alternatives

SECTION 10

DESCRIPTION OF MASTER PLAN CONFIGURATION AND ABATEMENT SCHEDULE

The result of the CSO Abatement Program is a Master Plan and abatement schedule derived from the analysis of the various alternatives. Such a schedule has been developed based on the phasing of the most applicable nonstructural, minimal structural, and structurally intensive alternatives as presented and discussed in Sections 8 and 9.

Those measures found to be most applicable to the reduction in discharge of combined sewer overflow to the Genesee River, Irondequoit Bay, and the Rochester Embayment of Lake Ontario are presented in Figure 62. The nonstructural alternatives are presented in the form of "Preparation and Implementation of Source Control Regulations" which include planning concepts to minimize percent imperviousness associated with future growth, the application of porous pavement in selected drainage areas, and the better utilization of surface storage and overland flow attenuation.

In addition to the application of source control measures, it is also recommended that a significant effort be extended in the area of system maintenance. It has been shown that the level of conveyance system maintenance is significant in reducing the impact of combined sewer overflows. Conveyance system maintenance not only alleviates flooding problems but also helps to eliminate or at least minimize the first-flush phenomenon.

It is also recommended that the extent (frequency and areal coverage) and effectiveness of street sweeping be increased. Optimization of street sweeping frequency and efficiency is an effective method in reducing the surface buildup of pollutants. Control over street cleaning frequency and efficiency can be an effective method for reducing the solids, toxicants, and oxygen demanding constituents contributed to the receiving waters via both combined sewer overflow and stormwater discharges.

Of the minimal structural alternatives evaluated, it was found that modest improvements to the St. Paul Boulevard Interceptor, selective regulator modifications, and the partial blockage of high impacting overflows could have a very significant effect on reducing the discharge of combined sewer overflows from the conveyance system relief points. It is projected that the removal of the three throttling constraints in the St. Paul Boulevard Interceptor will reduce the annual discharge of combined sewer overflow by 73 percent. Regulator modifications would have to be conducted in conjunction with the interceptor modifications in order to take advantage of the enhanced conveyance capacity. It is interesting to note that due to the attenuation of flow within the interceptor, the peak flow measured

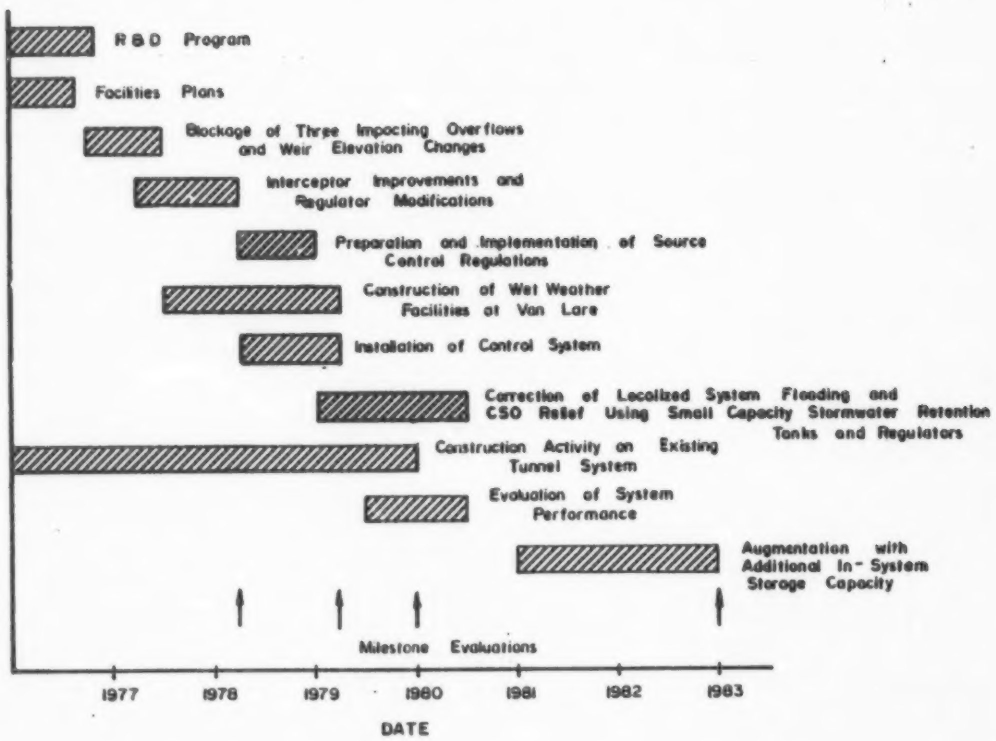

FIGURE 62. Rochester Combined Sewer Overflow Abatement Program Preliminary Implementation Plan

at the existing Van Lare STP dry-weather facilities would not be expected to increase beyond 270 cfs based on the 2 Yr-2 Hr design storm. The duration of the peak flow would, however, be expected to be tripled as a result of the modifications.

Other minimal structural alternatives evaluated and subsequently recommended for application include the addition of control structures and a control system to take optimum advantage of existing and proposed potential in-system storage. It is proposed that control structures be placed at the outlet of the Genesee Valley Interceptor, the proposed Genesee River Southeast Interceptor, and the Culver Goodman Tunnel complex. There is a potential storage capacity of approximately 80 mil gal which would not be utilized without the placement of the these control structures. Furthermore, the addition of control structures at selected locations within the surface conveyance system can also possibly reduce the frequency and quantity of overflow by utilizing the available storage in many of the smaller sewer conduits that are tributary to the main interceptor.

Upon implementation of control structures, an operating management tool is required to make the most intelligent use of the available storage during a wet-weather event. For this purpose, it is recommended that a control system be developed prior to or at the time of designing the required control structures. It is proposed that the concepts involved in the application of the SSM be incorporated as part of the predictive logic in the control system.

Of the structurally intensive alternatives, it is proposed that the high-rate flocculation and sedimentation facilities be constructed at the site of the present Van Lare STP. These facilities with a hydraulic capacity of 275 mgd and a design surface loading rate of 2000 gpd/ft^2 would have the capability of achieving a 63 percent average removal of total suspended solids for the design storm.

It is of most importance that the wet-weather treatment facilities be placed on line prior to the acceptance of combined sewer overflow discharges to the Cross Irondequoit Tunnel. The present dry-weather facilities at the Van Lare STP do not have sufficient hydraulic or process capacity to handle the total wet-weather flows anticipated at the Irondequoit Pumping Station.

It is also proposed that a number of the localized system flooding and CSO problems be corrected using relatively small capacity stormwater retention tanks. There are several areas identified in this report where this type of remedial solution may have application. The retention tanks must be applied to the retention of stormwater only and therefore only connected to the catchbasins with attenuated discharge to the trunk sewer. The rate of discharge from a stormwater detention tank would be determined by the head presented by the trunk sewer.

As shown in Figure 62 the implementation of the Master Plan should be well reviewed on a step-by-step basis to allow feed back as to the effectiveness of each abatement measure following application. Of particular importance is the evaluation of the need for additional in-system storage capacity which is proposed to be the last item to be implemented. The initial network modeling indicates that 62 mil gal of in-system storage capacity would be necessary to prevent discharge of system overflow resulting from the design storm. Many of the measures recommended prior to storage augmentation may further reduce the necessary storage requirements. It is for this reason that an extensive evaluation of system performance be required prior to the application of additional storage.

Figures 63 through 66 show the expected response of the quality of the Genesee River and Rochester Embayment of Lake Ontario to the implementation of the most applicable nonstructural, minimal structural, and structurally intensive alternatives as presented in Figure 61. The figures indicate that the quality of the River and of the Embayment can be substantially enhanced by the implementation of several nonstructural and minimal structural alternatives. These alternatives represent small capital expenditures relative to the benefits derived from their implementation.

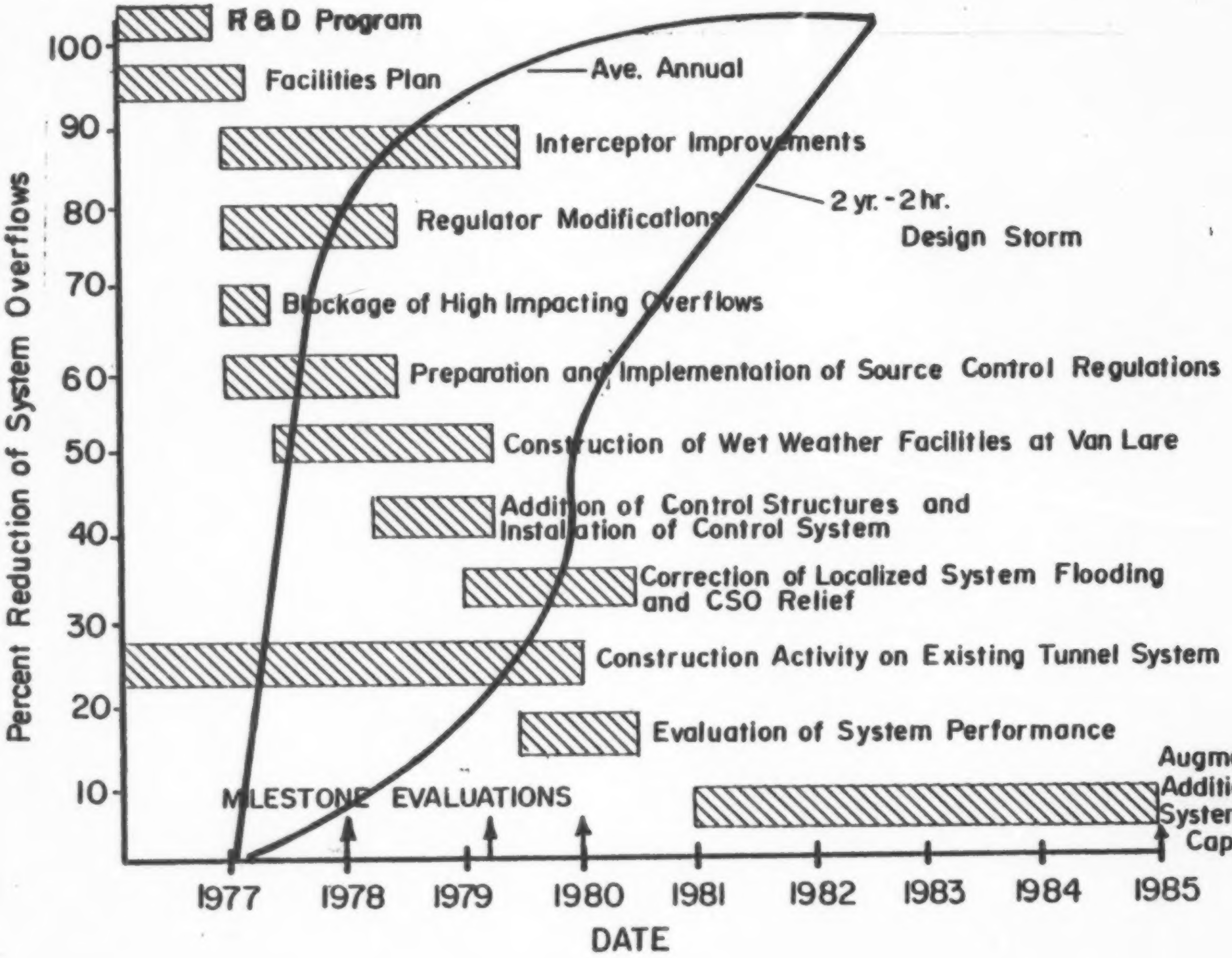

FIGURE 63. Total System Overflow Reduction Response to Implementation of Abatement Measures

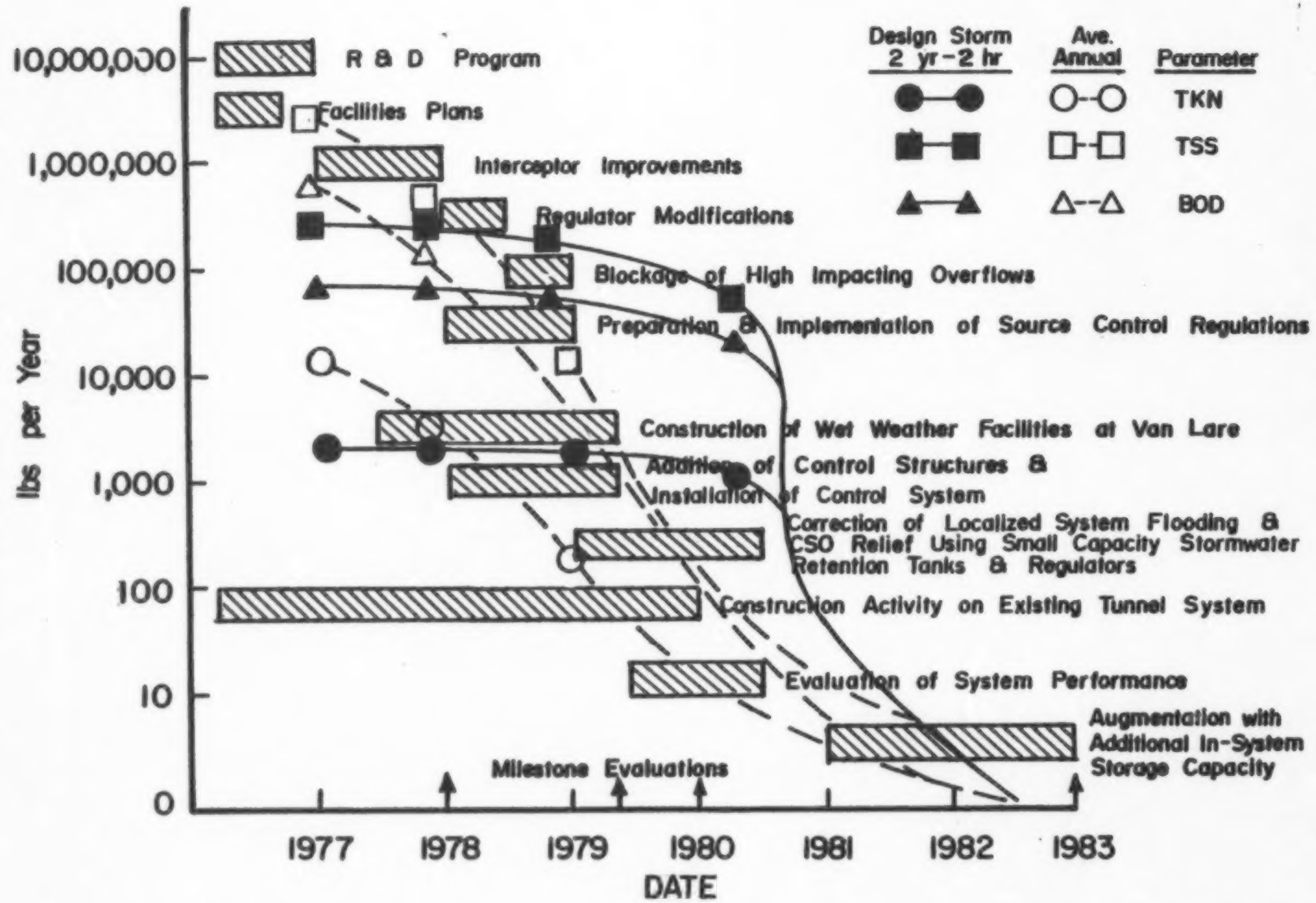

FIGURE 64. Total System Pollutant Loading Reduction to the Genesee River Response to Implementation of Abatement Measures

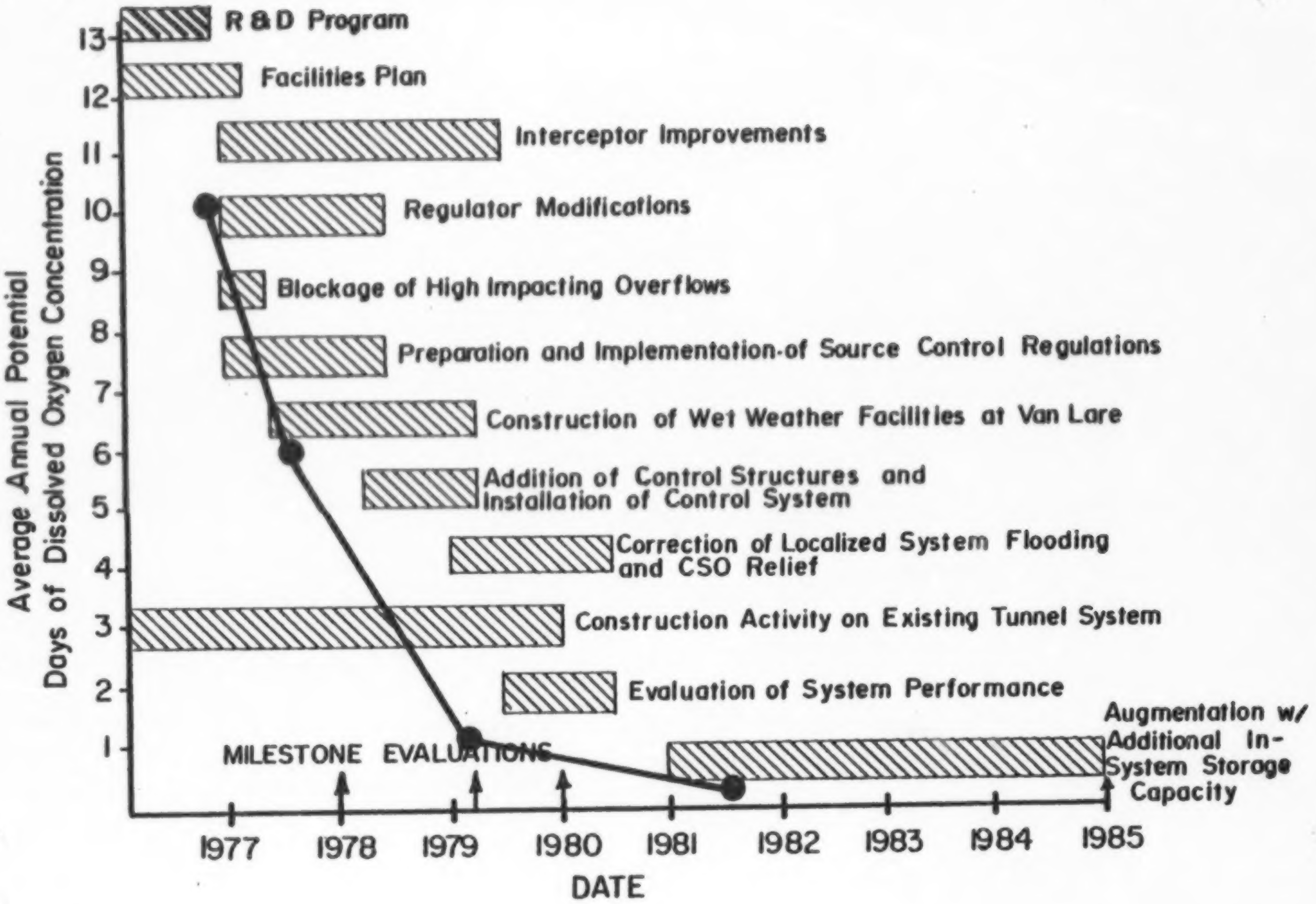

FIGURE 65. Genesee River Reduction in Potential Dissolved Oxygen Contravention Response to Implementation of Abatement Measures

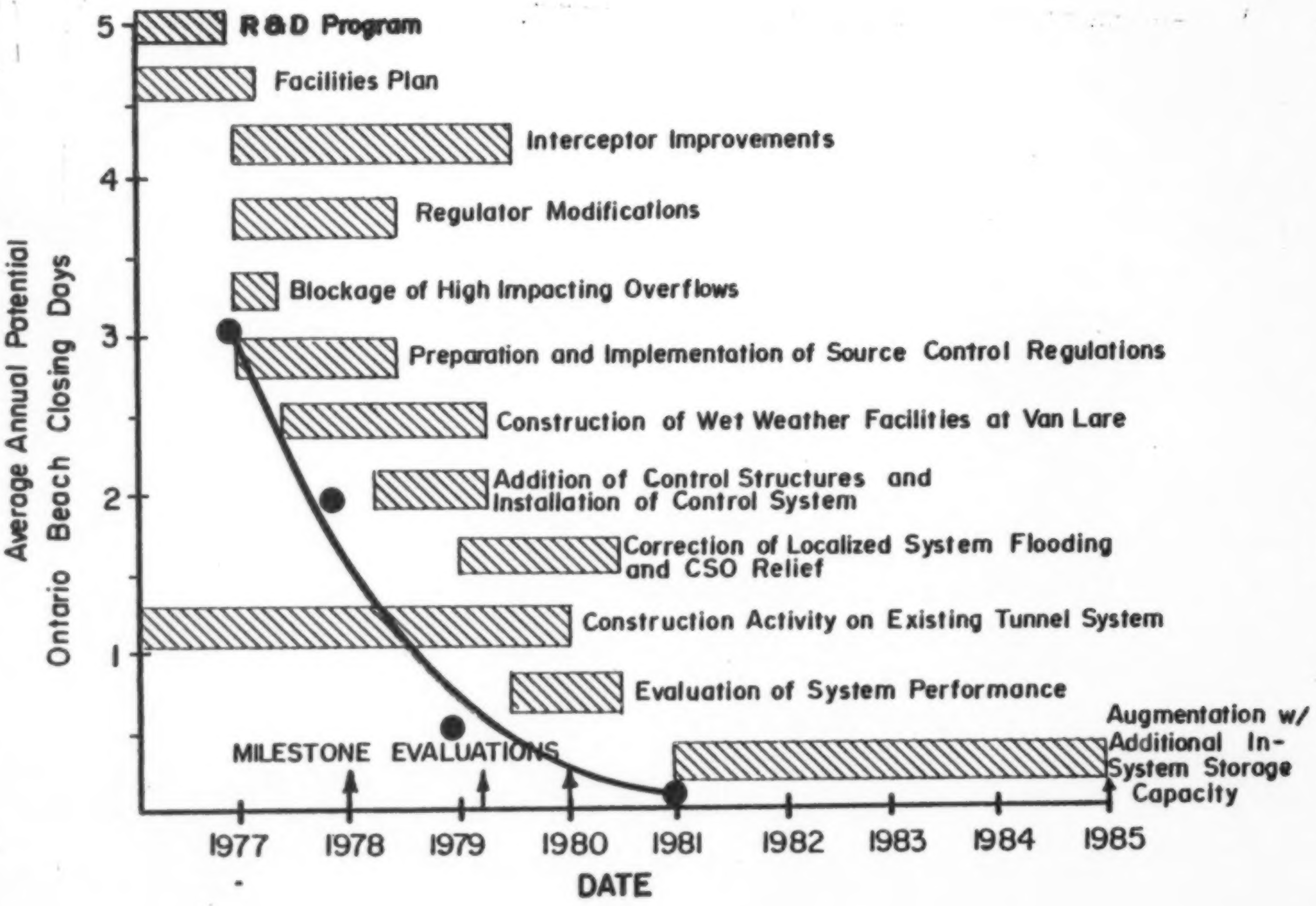

FIGURE 66. Reduction in Potential Ontario Beach Closing Days Response to Implementation of Abatement Measures

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